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ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

HARVEY WILSON DRIVE RECONSTRUCTION
WBS NO. N-000733-0002-3
HOUSTON, TEXAS

REPORTED TO
OMEGA ENGINEERS, INC.
HOUSTON, TEXAS

by

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EXECUTIVE SUMMARY

The report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the proposed Harvey Wilson Drive reconstruction (Key Map 494 L, M, Q, and R) in Houston, Texas. A vicinity map is presented on Plate A-1 in Appendix A. The total length of the project alignments is approximately 6,500 linear feet, which includes Harvey Wilson Drive from Lockwood Drive to Clinton Drive; Plastics Avenue from Harvey Wilson Drive to Armour Drive; Kress Street from the railroad crossing between Clinton Drive and Harvey Wilson Drive to Armour Drive; and Gazin Street from Clinton Drive to Armour Drive. Based on our investigation, the existing pavements along the project alignments consists of portland cement concrete (PCC) pavements, some of which are overlaid with asphaltic concrete (AC).

Details on the proposed sizes and invert depths of the underground utilities are not available at this time. However, AEC was informed that the invert depths of the utilities are expected to be between 10 to 15 feet below existing ground surface, and they will be constructed using open cut method and/or auger methods along the project alignments.

The objectives of our investigation are to conduct field investigation and laboratory testing to define subsurface conditions along the project alignments, and to provide geotechnical recommendations for the pavement reconstruction and the associate underground utilities installation. Environmental Site Assessment (ESA) is not within the scope of this report.

1. The subsurface conditions were investigated by drilling 14 soil borings to depths of 25 to 30 feet below existing pavements; Borings B-2 and B-10 were converted into piezometers PZ-1 and PZ-2, respectively. Based on the soil borings, the subsurface natural soils consists predominantly of firm to hard, interbedding Fat Clays (CH) and Lean/Sandy Lean Clays (CL). A layer of very soft Sandy Lean Clay (CL) with a SPT blow count of 2 blows/foot was found between depths of 13 and 19 feet in Boring B-13. A stratum of firm Silty Clay w/Sand (CL-ML) exists below a depth of about 6 feet in Boring B-7. Approximately 2 to 9 feet of medium dense Clayey Sand (SC) was encountered in Borings B-1, B-2 and B-8 at depth varies from 2 to 17 feet deep below existing ground surface. Approximately 3 to 12 feet of Silty Sand (SM) was encountered in Borings B-6, B-7, and B-10 through B-14 at depth varies from 5 to 18 feet below existing ground surface. The silty sand stratum found from a depth of about 4 to 13 feet was very loose to loose. Fill soils were encountered in Borings B-2, B-4, B-5, B-7, and B-9 through B-13 below existing pavement to depths ranging from

about 2 to 18 feet deep. The fill materials are mostly firm to hard Fat/Sandy Fat Clay (CH) and Lean Clay with Sand/Sandy Lean Clay (CL) and stiff Sandy-Silty Clay (CL-ML) soils. However, in Boring B-5, a thick layer of very soft to firm, Sandy Lean Clay (CL) fill was found between depths of about 4 and 18 feet. Ferrous and calcareous nodules and slickensides were encountered in the clay soils. Granular soils (gravel, sand, silt), clay soils with low plasticity, sand and silt seams/lenses in clayey soils as well as slickensides in fat clays may become unstable during excavation, especially when saturated. Such soils have a tendency to slough or cave in when not laterally confined, such as in unsupported excavations. The Contractor should be aware of the potential for cave-in of the soils.

2. Based on our borings, groundwater was encountered at depths ranging from 8 to 21 feet below existing grade during drilling and water level was subsequently rose to depth ranging from 6 to 15.3 feet 15 minutes after the initial encounter. This indicates the ground water is pressurized. Water level was measured at depths of about 9.7 and 9.6 feet in Piezometer PZ-1 on August 28, 2009 and on September 28, 2009, respectively. In Piezometer PZ-2 water level was measured at depths of about 5.1 and 5 feet on August 28, 2009 and on September 28, 2009, respectively. Detailed groundwater information is summarized in Section 4.4 of this report. The groundwater depth will fluctuate depending on seasonal rainfall and other climatic events. AEC recommends that the Contractor verify groundwater depths and flow rates before starting work to determine the presence of groundwater and potential pressurized aquifer and/seepage conditions, and the feasible groundwater control technique, if needed.

3. As part of our Phase I fault investigation, we evaluated fault locations based on a review of available literature, public maps, and documented active faults map of the project area. A review of published Principal Surface Faults of the Houston Central Metropolitan Area (after O'Neill and Van Siclen, with addition by C. Norman, 2004) indicates that the project alignment is located approximately one mile away to a small and unnamed SSW-NNE trending fault of Clinton Fault System. During our brief site reconnaissance, we did not encounter obvious physical evidence that would indicate possible fault movements within the project area; however, this does not rule out possibility of future movements of documented or yet to be identified fault(s) that may affect the project alignment.

4. No obvious evidence of hazardous materials was encountered in the recovered soil samples. However, the presence of potential hazardous material along other portions of the project alignments cannot be discounted based upon the limited quantity and number of samples taken in our investigation.

5. Geotechnical recommendations for the design and construction of the proposed PCC pavement, utilities installation by open-cut method and augering method, manholes construction, groundwater control and others associated constructions, are presented in Sections 5.

6. This Executive Summary provides a summary of the investigation and should not be used without the full text of this report.

ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

HARVEY WILSON DRIVE RECONSTRUCTION
WBS NO. N-000733-0002-3
HOUSTON, TEXAS

1.0 INTRODUCTION

1.1 General

The report submitted herein presents the results of Aviles Engineering Corporation's (AEC) geotechnical investigation for the proposed Harvey Wilson Drive reconstruction (Key Map 494 L, M, Q, and R) in Houston, Texas. A vicinity map is presented on Plate A-1 in Appendix A. The purpose of our investigation is to evaluate general subsurface soil and ground water conditions, and provide geotechnical recommendations for the design and reconstruction of roadway and associated underground utilities.

The total length of the project alignments is approximately 6,500 linear feet (see Plate A-2), which includes Harvey Wilson Drive from Lockwood Drive to Clinton Drive; Plastics Avenue from Harvey Wilson Drive to Armour Drive; Kress Street from the railroad crossing between Clinton Drive and Harvey Wilson Drive to Armour Drive; and Gazin Street from Clinton Drive to Armour Drive. Based on our investigation, the existing pavements of project alignment are Portland Cement Concrete (PCC) or Asphalt (AC) overlain PCC pavements.

Details on the sizes and invert depths of the underground utilities are not available at this time. However, AEC was informed that the invert depths are expected to be between 10 to 15 feet below existing ground surface and the utilities will be constructed using open cut method and/or auger methods. The objectives of our investigation are to investigation the subsurface conditions along the project alignment, and to provide geotechnical recommendations for the pavement reconstruction and the associate underground utilities installation.

1.2 Authorization

This work was authorized via an e-mail on August 13, 2009, by Mr. Richard G. Castaneda, P.E., President of Omega Engineering, Inc., and upon acceptance of AEC Proposal No. G2009-03-12R2, dated July 31, 2009.

1.3 Scope of Work

The scope of work for this investigation included performing field exploration, laboratory soil testing, and engineering analyses to develop recommendations for the construction of new pavements and associated underground utilities. Note that Environmental Site Assessment (ESA) is not within the scope of this geotechnical report.

2.0 SUBSURFACE EXPLORATION

2.1 Soil Borings

The subsurface exploration consisted of drilling and sampling a total of 14 borings along the project alignment, to depths ranging from 25 to 30 feet deep below existing grade. Borings B-5, B-8 and B-13 were drilled on Plastics Avenue, Kress Street and Gazin Street, between Harvey Wilson Drive and Armour Drive, respectively. The other borings were drilled along Harvey Wilson Drive between Lockwood Drive and Clinton Drive. The approximate boring locations are shown on Plates A-2a through A-2c in Appendix A. Total drilling footage is 355 feet. Borings B-2 and B-10 were converted into piezometers upon completion of drilling. Piezometer installation details are presented on Plates B-1 and B-2 in Appendix B.

2.2 Drilling and Sampling Methods

Drilling was conducted using a truck-mounted drilling rig; existing PCC pavements were cored through using a diamond-bit core barrel to facilitate our borings. The soil borings were advanced using dry continuous flight auger technique, then using wet rotary method once groundwater and/or caving soils were encountered. Undisturbed samples of cohesive soils were obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in general accordance with ASTM D-1587. Strength of the cohesive soils was estimated in the field using a hand penetrometer. Granular soils were sampled with a 2-inch split-barrel sampler in accordance with ASTM D-1586. Standard Penetration Test resistance (N) values were recorded for the granular soils as “Blows per Foot” and are shown on the boring logs. All borings (except the piezometers) were grouted with cement-bentonite and then patch with lean concrete upon completion of the drilling. Borings B-2 and B-10 were converted into piezometers, the piezometers were pulled and grouted after the 30-day water level reading. Well and Plugging Reports for the two piezometers, as reported to Texas Department of Licensing and Regulation (TDLR), are presented in Appendix E.

The undisturbed samples of cohesive soils were extruded mechanically from the core barrels in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study.

3.0 LABORATORY TESTING PROGRAM

Samples from the borings were examined and classified in the laboratory by a geotechnical technician under supervision of a geotechnical engineer. AEC's project engineer assigned laboratory tests on selected samples to evaluate the engineering properties of the subsurface soils. The tests included Atterberg limits, moisture content, percent passing No. 200 sieve, and dry unit weight; these tests were used to obtain index properties and confirm field soil classification. Unconfined compression and triaxial consolidated-undrained (UU) tests were performed on selected undisturbed samples to estimate the shear strength of cohesive soils. The laboratory test results are summarized on the boring logs on Plates A-3 through A-16, in Appendix A. The key to symbols, classification of soils for engineering purposes, terms used on boring logs, and ASTM designation for soil laboratory testing are presented on Plates A-17 through A-20, respectively, in Appendix A.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 Geologic Faults

As part of our Phase I fault investigation, we evaluated fault locations based on a review of available literature, public maps, and documented active faults map of the project area. A review of published Principal Surface Faults of the Houston Central Metropolitan Area (after O'Neill and Van Siclen, with addition by C. Norman, 2004) indicates that the project alignment is located approximately one mile away west of a small and unnamed SSW-NNE trending fault of the Clinton Fault System. During our brief site reconnaissance, we did not encounter obvious physical evidence that would indicate possible fault movements within the project area; however, this does not rule out possibility of future movements of documented or yet to be identified fault(s) that may affect the project alignment.

4.2 Hazardous Materials

No odor or physical evidence of hazardous materials or contaminants was observed in the recovered soil samples. However, the presence of potential hazardous material along other portions of the project alignments cannot be discounted based upon the limited quantity and number of samples taken in our investigation.

4.3 Surface and Subsurface Conditions

4.3.1 Existing Pavements

A summary of existing pavements encountered at the boring location is presented in Table 1 below. At various locations, the existing portland cement concrete (PCC) pavements have experienced mid-slab, corner and joint cracking and fractures, differential movements along joints. Some areas have been repaired with asphaltic concrete (AC) overlay.

TABLE 1. EXISTING PAVEMENT ENCOUNTERED IN SOIL BORINGS

Boring ID	Street(s)	AC (in)	PCC (in)
B-1	Harvey Wilson Dr.	1.6	8.9
B-2	Harvey Wilson Dr.	1.4	7.1
B-3	Harvey Wilson Dr.	-	6.8
B-4	Harvey Wilson Dr.	-	6.5
B-5	Plastics Ave.	1.0	6.0
B-6	Harvey Wilson Dr.	-	6.5
B-7	Harvey Wilson Dr.	-	7.0
B-8	Kress St.	1.4	7.1
B-9	Harvey Wilson Dr.	-	7.0
B-10	Harvey Wilson Dr.	1.3	7.7
B-11	Harvey Wilson Dr.	0.9	5.6
B-12	Harvey Wilson Dr.	-	8.0
B-13	Gazin St.	-	7.0
B-14	Harvey Wilson Dr.	-	8.0

4.3.2 Subsurface Soil Stratigraphy

Detailed subsurface soil information is presented in the individual boring logs on Plates A-3 through A-16 in Appendix A. The subsurface soil stratigraphy along the proposed project alignment is shown on the Generalized Soil Profile on Plates B-3 through B-5 in Appendix B.

Based on information from the soil borings, the subsurface natural soils consists predominantly of firm to hard, interbedding Fat Clays (CH) and Lean/Sandy Lean Clays (CL). A layer of very soft Sandy Lean Clay (CL) with a SPT blow count of 2 blows/foot was found between depths of 13 and 19 feet in Boring B-13. A stratum of firm Silty Clay w/Sand (CL-ML) exists below a depth of about 6 feet in Boring B-7.

Approximately 2 to 9 feet of Clayey Sand (SC) with high plasticity was encountered in Borings B-1, B-2 and B-8 between depths of about 2 and 19 feet. Approximately 3 to 12 feet of Silty Sand (SM) was encountered in Borings B-6, B-7, and B-10 through B-14 between depths of about 15 and 21 feet. The silty sand stratum found from a depth of about 4 to 13 feet was very loose to loose.

Fill soils were encountered in Borings B-2, B-4, B-5, B-7, and B-9 through B-13 below existing pavement to depths ranging from about 2 to 18 feet deep. The fill materials are mostly firm to hard Fat/Sandy Fat Clay (CH) and Lean Clay with Sand/Sandy Lean Clay (CL) and stiff Sandy-Silty Clay (CL-ML) soils. However, in Boring B-5, a thick layer of very soft to firm, Sandy Lean Clay (CL) fill was found between depths of about 4 and 18 feet. Ferrous and calcareous nodules and slickensides were encountered in the clay soils.

The cohesive soils encountered have Liquid Limit (LL) ranging from 21 to 67 and Plasticity Indices (PI) ranging from 7 to 50. This indicates that the cohesive soils have slight to very high plasticity.

The cohesive soils are classified as “CL-ML”, “CL” and “CH” type soils in accordance with the Unified Soil Classification System (USCS). “CH” type soils undergo significant volume changes due to seasonal changes in moisture contents. “CL” type soils with lower LL (less than 40) and PI (less than 20) generally do not undergo significant volume changes with changes in moisture content. However, “CL” soils with LL approaching 50 and PI greater than 20 essentially behave as “CH” soils and could undergo significant volume changes. Granular soils are classified as “SC” and “SM”. Granular soils and soils with PI approaching 10 and below are prone to loose their strength when they become saturated. Details of the soils encountered during drilling are presented in the boring logs.

4.4 Groundwater Conditions

Groundwater level information for each boring during and after drilling, as well as piezometer reading information, is presented in Table 2 below. The ground water depths and subsurface soil moisture contents vary with environmental variations such as frequency and magnitude of rainfall and the time of year when construction is in progress.

TABLE 2. GROUNDWATER DEPTHS BELOW EXISTING GROUND SURFACE

Boring No.	Groundwater Level Encountered During Drilling (ft)	Groundwater Level 15 Minutes After Initial Encounter (ft)	Groundwater Level in Piezometers (ft)	
			24 hour readings	30 day readings
B-1	12.3	10.6	N/A	N/A
B-2 (PZ-1)	14.4	10.5	9.7	9.6
B-3	13.3	11.0	N/A	N/A
B-4	20.3	15.3	N/A	N/A
B-5	21.0	15.3	N/A	N/A
B-6	9.0	8.0	N/A	N/A
B-7	11.0	7.5	N/A	N/A
B-8	8.0	7.2	N/A	N/A
B-9	16.5	9.9	N/A	N/A
B-10 (PZ-2)	9.0	6.0	5.1	5.0
B-11	11.5	8.8	N/A	N/A
B-12	12.6	9.9	N/A	N/A
B-13	9.0	N/A	N/A	N/A
B-14	11.0	9.3	N/A	N/A

Note: 24 hour readings and 30 day readings were measured on August 28, 2009 and September 28, 2009, respectively.

4.5 Subsurface Variations

The information contained in this report summarizes the conditions encountered on the dates the borings were drilled. Due to past construction along the project alignments and in the area, variations in the soil characteristics especially in the fill soils should be anticipated. The groundwater depths and subsurface soil moisture contents will vary with seasonal and environmental variations, such as tidal fluctuations, frequency and magnitude of rainfall and the time of year when construction is in progress.

Clay soils in the Houston area typically have secondary features such as slickensides and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples which were generally obtained at intervals of 2 feet in the top 10 feet and at intervals of 5 feet thereafter to the termination depth. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while some of AEC's logs show the soil secondary features, it should not be assumed that the features are absent where not indicated on the logs. The Contractor should be aware that the cohesive soils at the site may have planes of weakness, resulting in a tendency to cave in when not laterally confined.

5.0 GEOTECHNICAL ENGINEERING ANALYSIS AND RECOMMENDATIONS

This project calls for the reconstruction of pavements and associated utilities along Harvey Wilson Drive between Lockwood Drive and Clinton Drive, and connecting sections on Plastics Avenue, Kress Street and Gazin Street.

5.1 New PCC Pavement

AEC understands that the existing jointed PCC pavements (with AC overlay at various places) will be replaced with a new 42-ft wide jointed PCC roadway. Available traffic counts information indicated that the Harvey Wilson Drive at the intersecting Kress Street has highest traffic with 2001 and 2006 average daily traffic (ADT) of 1,570 and 2,310, respectively. Judging from the conditions of the existing PCC pavements with thicknesses ranging between about 6 and 9 inches, and the industrial setting of the project area, there are likely high incidents of heavy truck traffic on street pavements in this area.

For estimating the future traffic of the new PCC roadways for Harvey Wilson Drive and the connecting sections on Kress Steet, Plastics Avenue and Gazin Streets, AEC assumed an ADT growth rate of 10% from 2006 to 2011 (the base year), and a traffic growth rate at 10% from 2011 to 2016 to account for the projected traffic growth, and at 5% from 2016 through 2041 (30-year design life). Based on the assumed growth rate, the ADT in the base year of 2011 would be 3,732. Assuming 10 percent heavy trucks (FHWA Class 5 or greater) in this heavily industrial area, and a directional and lane distribution factor of 0.6 and 0.8, respectively, we estimated that the design traffic loading on the design lane is approximately equals to 4.45×10^6 repetitions of an 18-kip Equivalent Single-Axle Load (ESAL). The traffic load should be verified in the final design phase.

5.1.1 Pavement Thickness

The design procedure for determining concrete pavement thickness for rigid pavements was in accordance with AASHTOWARE DARwin 3.0 computer program. This program was developed based on the 1993 AASHTO Guide for Design of Pavement Structures, which was originally developed from the AASHTO Road Test. The following parameters were used in the design. Design input parameters should be confirmed prior to final design.

Overall Standard Deviation (S_o)	0.34
Reliability (R)	95%
Initial Serviceability (p_0)	4.5
Terminal Serviceability (p_t)	2.5
Joint Transfer Coefficient (J)	3.0
Drainage Coefficient (C_d)	1.0
Roadbed Soil Resilient Modulus (M_r)	2,000 psi for wet season 4,500psi for dry season
Composite Modulus of Lime Stabilized Soils (E_{SB})	30,000 psi
Mean Effective Composite Modulus of Subgrade Reaction (k)	79 pci
Modulus of Elasticity of Concrete (E_c)	3.37×10^6 psi
Mean Concrete Modulus of Rupture (S'_c)	600 psi (at 28 day)

Note: C.O.H. Specifications No. 02751 requires a min. of 5-1/2 sacks of cement per cubic yard, and design mix shall meet min. S'_c requirements of 500 psi and 600 psi at 7th and 28th day, respectively

Minimum Compressive Strength of Concrete (fc')	3,500 psi (at 28 day)

Based on the calculation results obtained by using AASHTOWARE™ DARWin™ 3.0 program, we recommend the following concrete pavement section:

TABLE 3. RECOMMENDED PAVEMENT SECTION THICKNESS

SECTION	SECTION THICKNESS (in)
Portland Cement Concrete, Jointed, Reinforced	10
Subbase: 7% lime-stabilized subgrade for fat clay subgrade soils	8

Note: The actual percentage of lime should be confirmed by laboratory testing prior to construction.

5.1.2 Subgrade Preparation

The pavement subgrade should be free of any vegetation, organics, and debris. The roadway excavation should be in accordance with Section 02315 of the 2009 City of Houston Standard Construction Specifications (COHSCS). Cleaning and grubbing should be in accordance with Section 02233 of the 2009

COHSCS. Any removal of tree stumps, vegetation and roots will create significant disturbance to the soils. It is important that these excavations be backfilled with compacted select fill. The exposed surface should be proof-rolled in accordance with Item 216 of the 2004 TxDOT Standard Specifications for Construction of Highways, Streets, and Bridges. The surface should be checked by qualified geotechnical professionals to identify any weak, soft or loose soils, vegetation, organics, and deleterious materials. Such unsuitable materials should be removed and replaced with moist compacted select fill. For the locations where fill is required to achieve the design grade, compacted select fills should be used to raise the grade. We recommend that after clearing, grubbing, and excavation to grade, a minimum 6 inches of the top subgrade soils be stabilized with at least 7 percent hydrated lime for the high plasticity fat clay subgrade soils found at this site. The stabilized soils should be placed in 8-inch loose lifts and compacted to at least 95 percent of the ASTM D-698 maximum dry density at a moisture content between optimum and 3 percent wet of optimum. Lime stabilization should be in accordance with or Section 02335 of the 2009 COHSCS.

5.1.3 Reinforcing Steel Requirements

The required longitudinal and transverse rebars for the concrete pavement can be computed using following equation:

$$A_s = FLW/(2f_s) \qquad \text{........... Equation (1)}$$

where: A_s = Cross-sectional area of steel per foot width of slab, sq. in.;
F = Coefficient of resistance between slab and subgrade, use 1.8;
L = Distance between free transverse joints or between free longitudinal edges, feet;
W = Weight of pavement per foot width of slab, lb/sq. ft.; and
f_s = Allowable working stress in the steel, psi; typically, a value equivalent to 75 percent of steel yield strength is used for working stress, for Grade 60 steel, use 45,000 psi.

Based on the above recommended pavement thickness and reinforcement requirements, the required reinforcing steel for a 10-inch thick concrete pavement (with an expansion joint spacing of 80 feet) is as follows:

TABLE 4. REINFORCED STEEL DESIGN FOR RIGID PAVEMENTS

PAVEMENT WIDTH (feet)	STEEL SIZES AND CORRESPONDING MAXIMUM SPACING (center to center, inches)	
	LONGITUDINAL	TRANSVERSE
42	#4 @ 12.9	#4 @ 24.6
	#5 @ 20.2	#5 @ 36

The layout of the reinforcing steel of selected size should be such that the end spacing and interior center-to-center spacing should be meet the maximum spacing criteria presented in Table 4.

The size and spacing of dowels to be provided at expansion joints for 6-inch thick concrete pavement are presented below. The detail should be in accordance with COHSCS Drawing No. 02752-01

TABLE 5. DOWEL SIZE AND SPACING FOR RIGID PAVEMENTS

PORTLAND CEMENT CONCRETE THICKNESS (inches)	DOWEL SIZE AND SPACING		
	DIAMETER (in.)	LENGTH (in.)	SPACING (in.)
10	1-¼	18	12

5.2 Underground Utilities Installation Using Open Cut Method

5.2.1 General

The proposed utilities invert depths were not available when this report was prepared, but anticipated to be between 10 to 15 feet deep below existing ground surface. According to the Client, we understand that some sections of the proposed underground utilities along the project alignments will be installed using either open-cut or augering method.

Based on our borings and the anticipated invert depths of proposed utilities, the construction will likely encountered different soils including firm to hard fat and lean clays, medium dense to dense clayey sands, loose to dense silty sands. In Boring B-5, a layer of very soft to firm sandy lean clay fill was found between depth of about 4 and 18 feet. In Boring B-13, a thick, very loose to loose silty sand stratum exists between depths of about 4 and 13 feet, below which a very soft sandy lean clay stratum exists to a depth of about 19 feet.

Based on our groundwater observations in soil borings during drilling and piezometer readings, excavations below 5 feet may encounter groundwater.

5.2.2 Geotechnical Parameters

A summary of the recommended typical geotechnical parameters for different types and in-situ strengths of soils is presented on Plate C-1 in Appendix C. These values are based on the results of field and laboratory test data as well as our experience. It should be noted that because of the nature of the soil stratigraphy, parameters at locations away from the borings may vary substantially from values reported in the plates.

5.2.3 Trench Stability

The Contractor should be responsible for designing, constructing and maintaining safe excavations. The excavations should not cause any distress to existing structures.

Trenches Deeper than 20 Feet: OSHA requires that shoring or bracing for trenches deeper than 20 feet be designed by a licensed professional engineer.

Trenches 20 Feet Deep or Less: Trench excavations that are 20 feet deep or shallower may be shored, sheeted and braced, or laid back to a stable slope for the safety of workers, public and adjacent structures, except excavations which are less than 5 feet deep and verified by a competent person to have no cave-in potential. The excavation and trenching should be in accordance with Occupational Safety and Health Administration (OSHA), Safety and Health Regulations, 29 CFR, Part 1926. OSHA Soil Types for on-site soils are presented on Plate C-2 in Appendix C.

It should be noted that the soil types shown are based on the soil conditions encountered at the time of our investigation. Since OSHA soil types for cohesive soils are based on their shear strength and other site conditions, the soil types may be different during construction if soil conditions differ from those encountered during our investigation.

5.2.4 Critical Height

Critical Height is defined as the height a slope will stand unsupported for a short time; in cohesive soils, it is used to estimate the maximum depth of open-cuts at given side slopes. Critical Height may be calculated based on the soil cohesion. Values for various slopes and cohesion are shown on Plate D-1 in Appendix D. Cautions listed below should be exercised in use of Critical Height applications:

1. No more than 50 percent of the Critical Height computed should be used for vertical slopes. Unsupported vertical slopes are not recommended where granular soils or soils that will slough when not laterally supported are encountered within the excavation depth.

2. If the soil at the surface is dry to the point where tension cracks occur, any water in the crack will increase the lateral pressure considerably. In addition, if tension cracks occur, no cohesion should be assumed for the soils within the depth of the crack. The depth of the first waler should not exceed the depth of the potential tension crack. Struts should be installed before lateral displacement occurs.

3. Shoring should be provided for excavations where limited space precludes adequate side slopes, e.g., where granular soils will not stand on stable slopes and/or for deep open cuts.

4. All excavation and shoring should be designed and constructed by qualified professionals in accordance with OSHA requirements.

Plate D-2 in Appendix D presents the maximum allowable slopes in Soil Types A, B, and C for excavations less than 20 feet. If limited space is available for the required open trench side slopes, the space required for the slope can be reduced by using a combination of bracing and open cut as illustrated on Plate D-3 in Appendix D.

5.2.5 Computation of Bracing Pressures

The following method can be used for calculating earth pressures against temporary bracing. Lateral pressure resulting from construction equipment, traffic loads, or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure, if any, should also be considered. The active earth pressure at depth z can be determined by following equation, typical design soil parameters can be derived by determining the soil types and strength parameters found in individual boring logs and referring to Plate C-1 in Appendix C.

$$P_a = (q_s + \gamma h_1 + \gamma' h_2)K_a - 2c\sqrt{K_a} + \gamma_w h_2 \qquad \text{...........Equation (2)}$$

Where:

P_a	=	active earth pressure, pound per square feet (psf);
q_s	=	uniform surcharge pressure, psf;
γ, γ'	=	wet unit weight and buoyant unit weight of soil, pound per cubic feet (pcf);
h_1	=	depth from ground surface to ground water table, feet;
h_2	=	$z-h_1$, depth from ground water table to the point under consideration, feet;
z	=	depth below ground surface for the point under consideration, feet;
K_a	=	coefficient of active earth pressure;
c	=	cohesion of clayey soils, psf; and
γ_w	=	unit weight of water, 62.4 pcf.

Pressure distribution for the practical design of struts in open cuts for clays and sands are illustrated on Plates D-4 through D-6 in Appendix D. If excavations are located close to existing structures, we recommend using the coefficient of at-rest earth pressure (K_o) for design to reduce the potential for distress to the existing structures. Values of K_0 can also be found on Plate C-1 in Appendix C.

5.2.6 Bottom Stability

In open cuts, it is necessary to consider the possibility of the bottom failing by heaving, due to the removal of the weight of excavated soil. Heaving typically occurs in soft plastic clays when the excavation depth is sufficiently deep enough to cause the surrounding soil to displace vertically due to bearing capacity failure of the soil beneath the excavation bottom, with a corresponding upward movement of the soils in the bottom of the excavation. In fat and lean clays, heave normally does not occur unless the ratio of Critical Height to Depth of Cut approaches one. In very sandy and silty lean clays and granular soils, heave can occur if an artificially large head of water is created due to installation of impervious sheeting while bracing the cut. This can be mitigated if groundwater is lowered below the excavation by dewatering the area. Guidelines for evaluating bottom stability in clay soils are presented on Plate D-7 in Appendix D.

If the open cut excavation extends below groundwater, and the soils at or near the bottom of the excavation are mainly sands or silts or low plasticity clays, the bottom can fail by blow-out (boiling) when a sufficient hydraulic head exists. The potential for boiling or in-flow of granular soils increases where the groundwater is pressurized. To reduce the potential for boiling of excavations terminating in granular soils below pressurized ground water, the ground water table should be lowered and maintained at least 3 feet below the excavation. In extreme conditions, structural or chemical stabilization of the granular soils may be required.

If the braced excavation terminates in a cohesive soil, but is underlain by granular soils and is subject to hydraulic pressure, the factor of safety against bottom failure can be conservatively calculated (neglecting shear strength) from the equation presented below:

$$F = \gamma_S h_s / \gamma_w h_w \qquad \text{.......... Equation (3)}$$

Where
F = safety factor against blow-out, minimum 1.25;
γ_s = unit weight of cohesive soil above sand layer, pcf;
h_s = height of cohesive soil above sand layer, feet;
γ_w = unit weight of water, pcf; and
h_w = hydrostatic head, feet.

The most effective means of improving the bottom stability is to lower the groundwater table to at least 3 feet below the bottom of the braced excavation.

Secondary features such as calcareous and ferrous nodules, silt and sand seams, slickensides were encountered within cohesive soil in our borings. These secondary features may become sources of localized instability when they are exposed during excavation, especially when they become saturated. Such soils have a tendency to slough or cave in when not laterally confined, such as in auger pit excavations. The Contractor should be aware of the potential for cave-in of the soils. Low plasticity soils will lose strength and may behave like granular soils when saturated.

5.2.7 Loading on Buried Pipes

Earth Loads: For underground conduits to be installed using the open-cut trench method, the vertical soil load W_e can be calculated as the larger of the two values from Equations (4) and (6):

$$W_e = C_d \gamma B_d^2 \quad \text{........... Equation (4)}$$

$$C_d = [1- e^{-2K\mu'(H/B_d)}]/(2K_\mu') \quad \text{........... Equation (5)}$$

$$W_e = \gamma B_c H \quad \text{........... Equation (6)}$$

Where:
- W_e = trench fill load, pounds per linear foot (lb/ft);
- C_d = trench load coefficient, presented on Plate C-3, Appendix C;
- γ = effective unit weight of soil over the conduit, pcf;
- B_d = trench width at top of the conduit < 1.5 B_c, feet;
- B_c = outside diameter of the conduit, feet; and
- H = variable height of fill, feet;
  when the height of fill above the top of the conduit H_c >2 B_d, $H = H_h$ (height of fill above the middle of the conduit). When H_c < 2 B_d, H varies over the height of the conduit; and
- K_μ' = 0.1650 maximum for sand and gravel;
  0.1500 maximum for saturated top soil;
  0.1300 maximum for moist clay; and
  0.1100 maximum for saturated clay.

When the underground conduits are located below the ground water level, the total vertical dead loads should include the weight of the projected volume of water above the conduits.

Traffic Loads: The vertical stress p_L (psf) resulting from traffic loads can be obtained from Plate C-4 in Appendix C. The live load on the top of the underground conduit can be calculated from following equation:

$$W_L = p_L B_c \qquad \text{........... Equation (7)}$$

Where: W_L = live load on the top of the conduit, lb/ft;
p_L = vertical stress (on the top of the conduit) resulting from traffic loads, psf; and
B_c = outside diameter of the conduit, feet.

Lateral Loads: The lateral soil pressure p_l can be calculated from following equation; hydrostatic pressure should be added, if applicable.

$$p_l = 0.5\,(\gamma H_h + p_s) \qquad \text{........... Equation (8)}$$

Where: H_h = height of fill above the center of the conduit, feet;
γ = effective unit weight of soil over the conduit, pcf; and
p_s = vertical pressure on conduit resulting from traffic and/or construction equipment, psf.

5.2.8 Bedding and Backfill

Excavation and backfill for utilities should be in accordance with Sections 02317 and 02320 of the 2009 COHSCS. In general, bedding and backfill for the underground utilities should be in accordance with COHSC Detail Drawings, Drawing No. 02317-01, 02317-02, 02317-03 and 02317-08, as applicable.

Cohesive soils except CL-ML, with a LL of less than 45 and a PI in the range of 8 to 20 can also be used as backfill material. Backfill should be placed in loose lifts not exceeding 8 inches in thickness. Backfill within 3 feet of structures should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors. The fill should be compacted to a minimum 95% of the ASTM D-698 (Standard Proctor) maximum dry unit weight and at moisture content between optimum and 3% wet of optimum, in accordance with Section 02317 of the 2009 COHSCS.

5.3 Underground Utilities Installation Using Augering Method

5.3.1 General

We understand that some of the proposed underground utilities, especially water lines, will be installed by auger methods. The information in this report should be reviewed so that necessary ground water control and appropriate augering equipment and techniques can be planned and factored into the construction plan and cost estimate.

Augering operations should be performed in accordance with “Augering Pipe and Conduit”, Section 02447 of the 2009 COHSCS ". The Contractor is responsible for selecting, designing, installing, maintaining and monitoring safe augering systems and retaining professionals who are qualified and experienced to perform the tasks and who are capable of modifying the system, as required. The following discussion provides general guidelines to the Contractor for augering methods.

5.3.2 Auger Pits

Auger pits are required for jacking and receiving pipes. They should be designed and constructed in accordance with Item 3.04, Section 02447 of the 2009 COHSCS. Auger pits need to be shored because the side walls are normally cut vertically to conserve space. They should be designed as braced excavations in accordance with requirements in Section 5.2 of this report. The Contractor is responsible for designing, constructing and maintaining safe excavations. The excavations should not cause any distress to existing structures.

Hydrostatic Uplift Resistance For resistance against hydrostatic uplift, buoyant unit weights of 90 for concrete and 60 pcf for soil should be used. Minimum recommended safety factors against uplift are 1.1 for concrete weight, 1.5 for soil weight and 3.0 for soil friction. The design criteria for uplift resistance are presented on Plate D-8 in Appendix D.

Bottom Stability Bottom stability for open-cut method is presented in Section 5.2.6 of this report.

5.3.3 Reaction Walls

Reaction walls (if used) will be part of the auger pit walls; they will be rigid structures and support augering operations by mobilizing passive pressures of the soils behind the walls. The passive earth pressure can be calculated using Equation (9); we recommend that a factor safety of 2.0 be used for passive earth pressure. The typical design soil parameters are presented on Plate C-1 in Appendix C.

$$p_p = \gamma z K_p + 2cK_p^{1/2} \quad \text{........... Equation (9)}$$

Where,
- p_p = passive earth pressure, psf;
- γ = wet unit weight of soil, pcf;
- z = depth below ground surface for the point under consideration, feet;
- K_p = coefficient of passive earth pressure; and
- c = cohesion of clayey soils, psf.

Due to variation, soils with different strengths and characteristics will likely be encountered at a given location. The soil resulting in the lowest passive pressure should be used for design of the walls. The soil conditions should be checked by geotechnical personnel to confirm the recommended soil parameters.

5.3.4 Loadings on Pipes

Earth Loads For underground conduits to be installed by augering method, the vertical soil load can be calculated using Equations (10) and (11):

$$W_t = C_t \gamma B_t^2 - 2cC_t B_t \quad \text{........... Equation (10)}$$

$$C_t = [1- e^{-2K\mu(H/Bt)}]/(2K_\mu) \quad \text{........... Equation (11)}$$

Where:
- W_t = earth load under tunneled conditions, lb/ft;
- C_t = load coefficient, dimensionless, presented on Plate C-3 in Appendix C;
- γ = effective unit weight of soil over the pipe, pcf;
- B_t = maximum width of tunnel excavation, feet;
- H = height of cover above the top of the pipe, feet;
- c = cohesion of the soil above the excavation, psf;
- e = base of natural logarithms; and
- K_μ = 0.1650 maximum for sand;
  0.1500 maximum for saturated top soil;
  0.1300 maximum for moist clay; and
  0.1100 maximum for saturated clay.

When the underground conduits are located below the ground water level, the total vertical dead loads should include the weight of the projected volume of water above the conduits.

Traffic Loads The calculation of traffic loads on the pipes can refer to Section 5.2.7 of this report.

5.3.5 Auger Face Stability during Construction

General: The stability of an auger face is governed primarily by ground water and subsurface soil conditions. Based on our borings and the proposed utility invert depths, we anticipate that the majority of the utility construction will be advanced in firm to very stiff fat and lean clays, silty clays, with occasional strata of clayey sands or silty sands. In Boring B-5, a thick layer of very soft to firm sandy lean clay fill was found between depths of about 4 and 18 feet. Also, augering deeper than 5 feet will likely encounter groundwater. Secondary features such as slickensides, sand or silt seams/pockets/layers were also encountered within the cohesive soils, and could be significant at some locations. In addition, the type and property of subsurface soils are subject to change between borings, and may be different at locations away from our borings.

If granular soils are encountered during construction, the auger face may become unstable. Granular soils below ground water will tend to flow into the excavation hole; granular soils above the ground water level will generally not stand unsupported but will tend to ravel until a stable slope is formed at the face with a slope equal to the angle of repose of the material in a loose state. Thus, granular soils are generally considered unstable in an unsupported excavation face; uncontrolled flowing soil can result in large loss of ground.

Anticipated Ground Behavior: A Stability Factor $N_t = (P_z - P_a)/C_u$ may be used to evaluate the stability of an unsupported tunnel face in cohesive soils, where P_z is the overburden pressure to the tunnel centerline; P_a is the equivalent uniform interior pressure applied to the face; and C_u is the undrained shear strength of the soil. Generally, a Stability Factor (N_t) of 4 or less is desirable as it represents a practical limit below which tunneling may be accomplished without significant difficulty. Higher N_t values usually lead to large deformations of the soil around the tunnel, and increased subsidence. It should be noted that the exposure time of the face is most important; with time, creep of the soil will occur, resulting in a reduction of shear strength. The N_t values will therefore increase when construction progress rate is slow.

No size and invert depth information on the proposed utilities is available at this time. For estimating N_t values for utiltities construction along the project alignment, we assumed three pipe diameters of 12-, 36-and 48-inch, and invert depths of 5, 10 and 15 feet below existing grade. Using the soil boring information, we estimated N_t values of 2 or less for auger excavation within the firm to hard cohesive soils with the assumed pipe diameters and auger depths. However, N_t values are estimated to exceed 4 for auger excavations deeper than 5 feet in the thick and very soft clay soils found in Boring B-5, in the very loose to loose silty sand and very soft sandy lean clays found in Boring B-13, and at other locations where augering will advance through granular soils or clay soils with minimal or no cohesion. As noted earlier in this report, some of the cohesive soils have secondary features; those soils have a tendency to slough or cave in when not laterally confined. If significant saturated sand/silt seams are encountered during augering, casing/liner or pressure balancing should be promptly installed/implemented to enhance stability of the auger excavation.

Influence of Augering on Existing Structures: Disturbance and loss of ground from auger operations may create surface soil disturbance and subsidence, which in turn may cause distress to existing structures or pavements located in the zone of soil disturbance. The magnitude of the surface settlement and soil subsidence is proportionate to the volume of the excavation.

If existing foundations are located too close to the proposed augering alignment, subsidence or settlement of the soils adjacent to these foundations can result and cause unacceptable movements/distress in the existing foundations. Also, lateral movements of the soils away from the foundations (as a result of collapse/cave-in or subsidence of the soils above the tunnel) can reduce the foundation bearing capacity due to loss of overburden. Existing structures and foundations (located inside the influence zone) should be adequately protected before beginning tunneling operations. The influence zone of a auger hole is assumed to extend a distance of about 2.5i from the center of the auger hole, as shown on Plate D-9 in Appendix D.

With the assumed pipe diameters and invert depths, we estimated the resulting influence zones (extending from the centerline of the tunnel) to be approximately 3 to 9 feet for most of the typical soils encountered in our borings. Assuming good augering technique, we anticipate that the surface subsidence to be less than one inch in auger excavations in the most of the subsurface soils found in our borings. However, settlements caused by auger excavations in the thick, very soft to soft clay found in Boring B-5, and the very loose to loose silty sand found in Boring B-13, will be much higher.

Methods to mitigate movements and/or distress to existing structures will require the services of a specialty contractor. We emphasize that the size of the influence zone of an auger hole is difficult to determine because several factors influence the response of the soil to tunneling operations including type of soil, ground water level, type of tunneling equipment, method of tunneling, experience of operator and other construction in the vicinity. The values of tunnel influence zone presented herein are therefore rough estimates.

Measures to Reduce Distress from Augering: Impact to the existing foundations and structures can be mitigated by following proper augering procedures. Some methods to mitigate movement and/or distress to existing structures include supporting the excavation with the steel or concrete pipe as soon as the excavation is advanced and at short intervals, and properly grouting of the annular spaces where necessary, in accordance with Item 3.07, Section 02447 of the 2009 COHSCS.

To reduce the potential for the augering operation to influence existing foundations or structures, we recommend that the outer edge of the influence zone of the auger hole to be a minimum of 5 feet from the outer edge of the bearing zone of existing foundations. The bearing zone of existing foundation is defined by a line drawn downward from the outer edge of an existing foundation and inclined at an angle of 45 degree to the vertical.

For the thick, very soft to soft clay soils and the very loose to loose silty sand found in Borings B-5 and B-13, the areal extent of the weak soils may be probed during construction. If feasible and economical, the weak and soft soils may be reworked (with possible lime stabilization) and recompacted, or, removed and replaced with compacted select fill to increase stability and mitigate settlements. Adequate shoring, face support, and good groundwater control (where needed) should be provided for excavations and augering conducted within these weak soils.

Monitoring Existing Structures: Existing structures in the vicinity of the proposed auger alignment should be closely monitored prior to, during and for a period after augering operations. Several factors (including soil type and stratification, construction methods, weather conditions, other construction in the vicinity, construction personnel experience and supervision) may impact ground movement in the vicinity of the alignment. We therefore recommend that the Contractor be required to survey and adequately document condition of existing structures in the vicinity of the proposed alignment. The monitoring program for the proposed tunneling operations should be in accordance with Item 3.10, Section 02447 of the 2009 COHSCS.

5.4 Deflection of Flexible Pipes

Deflection is one of the controlling factors in the design of buried flexible or semi-rigid pipes, such as steel or ductile iron pipes. These pipes deflect under soil and surcharge loads; the amount of deflection is a function of the service load on the pipe, the stiffness of the pipe, and the surrounding soil.

The deflection can be calculated using the Modified Iowa Formula, expressed as Equation (12), and the effective stiffness, E' of the surrounding soil. The E' is a combination of the stiffness of the pipe embedment material, E'_B and the stiffness of the native soil within pipe embedment zone, E'_N. Long-term deflection values are typically used for flexible/semi-rigid pipe design; these values may be obtained by applying an appropriate deflection lag factor, D_L, to the short-term deflection values used in the Modified Iowa Formula.

$$\Delta x = \frac{D_L K W}{\frac{EI}{R^3} + 0.061E'} \quad \text{........... Equation (12)}$$

Where: Δx = pipe deflection, inch;

D_L = deflection lag factor, use 1.2 for granular backfill in accordance with "Steel Pipe and Fittings for Large Diameter Water Lines", Section 02518 of the 2009 COHSCS;

K = bedding constant, use 0.1 per Section 02518 of the 2009 COHSCS;

W = [W_e (from Eq. 6 for open cut/Eq. 10 for augering method) + W_L (from Eq. 7) +W_w], total service load on the crown of the pipe, lb per linear inch; W_w = weight of water prism (if any) above the crown of the pipe;

E = initial modulus (Young's modulus) of the pipe material, psi;

I = pipe wall moment of inertia per unit length, $in.^4/in.$;

R = mean pipe radius, inch; and

E' = effective modulus of soil reaction, psi.

The effective modulus of soil reaction, $E^{/}$, may be obtained from the equations presented below:

$$E' = zeta * E'_B \quad \text{........... Equation (13)}$$

$$zeta = \frac{1.44}{f + (1.44 - f) * E_B'/E_N'} \quad \text{........... Equation (14)}$$

Where:

$$f = \frac{B_d / B_c - 1}{1.154 + 0.444(B_d / B_c - 1)} \quad \text{........... Equation (15)}$$

B_d = trench width at the top of the pipe; and

B_c = outside diameter of the pipe.

For the stiffness of the pipe embedment material E'_B, 2,000 psi can be used for granular materials such as clayey sand, silty sand, silty gravel or clayey gravel (containing less than 12% fines) with minimum 95% ASTM D698 compaction. The stiffness of the native soil within the embedment zone, E'_N can be estimated in accordance with the soil parameters on Plate C-1 in Appendix C (in conjunction with the subsurface information from applicable boring logs), and from Table 6 below. Predictions of pipe deflection should be based upon the lowest soil stiffness category found at the elevation of the proposed pipe embedment zone.

TABLE 6. EFFECTIVE MODULUS OF SOIL REACTION E'_N

Cohesive Soils		Granular Soils		Stiffness Category	E'_N (psi)
Undrained Shear Strength (tsf)	Description	SPT (blows/ft)	Description		
> 3.0	Very Hard	> 50	Very Dense	A	3000
2.0 to 3.0	Hard	31 to 50	Dense	B	2000
1.0 to 2.0	Very Stiff	16 to 30	Medium Dense	C	1000
0.5 to 1.0	Stiff	8 to 15	Loose to Medium Dense	D	600
0.25 to 0.5	Firm	5 to 7	Loose	E	300
0.125 to 0.25	Soft	2 to 4	Very Loose	F	200
< 0.125	Very Soft	< 2	Very Loose	G	100

5.5 Pipe Systems Thrust Restraint

Thrust forces are generated in pressure pipes as a result of changes in pipe diameter, pipe direction or at the termination point of the pipes. The pipes could disengage at the joints if the forces are not balanced and if the pipe restraint is not adequate. Various methods of thrust restraint are used including thrust blocks, restrained joints, encasement and tie-rods.

Thrust restraint design procedures based on the American Water Works Association Manual M9 (1995) Concrete Pressure Pipes are discussed herein. Plate D-10 shows the force diagram generated by flow in a bend in a pipe and also gives the equation for computing the thrust force. An example computation of a thrust force for a given surge pressure and a bend angle is presented on Plate D-11, in Appendix D.

<u>Frictional Resistance:</u> The unbalanced force due to changes in grade and alignment can also be resisted by frictional force F_R, between the pipe and the surrounding soil. The resisting frictional force per linear foot of pipe against soil is computed as Equation (16):

$$F_R = f\,(2W_e + W_w + W_p) \qquad \text{........... Equation (16)}$$

Where, f = Coefficient of friction between pipe and soil

W_e = Weight of soil over pipe, lb/ft.;
W_w = Weight of water inside the pipe, lb/ft.; and
W_p = Weight of pipe (lb/ft)

The value of the frictional resistance depends on the material in contact with the backfill and the soil used in the backfill. For a steel or concrete pipe with compacted granular backfill, an allowable coefficient of friction of 0.3 can be used. To account for submerged conditions, a soil unit weight of 60 pcf should be used to compute the weight of compacted backfill on the pipe. Passive resistance of soil should not be included.

5.6 Manholes

Access manholes or manways may be constructed at selected locations along the alignment of the proposed utilities, at depths of about 10 and 15 feet below existing grade. The Contractor should be responsible for designing, constructing and maintaining safe excavations for the proposed manholes. Manhole open-cut excavations shall be in accordance with Section 5.2 of this report. Geotechnical recommendations to guide design of these below-grade structures are presented below.

5.6.1 Allowable Bearing Capacity

We assume mat foundation on prepared foundation soils/bedding will be used for the proposed manholes. The following net allowable bearing capacities, whichever is critical, may be used for proportioning the proposed mat foundations:

TABLE 7. ALLOWABLE NET BEARING CAPACITY FOR MANHOLE FOUNDATIONS

Applicable Soil Borings	Manhole Depth below Existing Grade (ft)	Net Allowable Bearing Capacity, psf	
		Sustained Load Only (F.S.=3.0)	Total Load (F.S.=2.0)
At/Near Boring B-5 and B-13	All	400	600
Other Locations	Above 5	750	1,125
	5 to 10	1,000	1,500
	10 to 15	1,500	2,250
	15 to 20	2,000	3,000

AEC recommends that soft cohesive soils or weak granular or low plasticity foundation soils be removed to a minimum depth of one foot below the mat foundations and replaced with cement-stabilized sand in accordance with Section 02321 of the 2009 COHSCS or compacted gravel wrapped with geofabric.

The net bearing pressure is determined by:

1. Summing the weight of the load applied to the foundation, the weight of the foundation and the weight of soil backfill placed above the foundation;
2. Subtracting the weight of soil excavated from the foundation; and
3. Dividing the result of items 1 and 2 by the base area of the foundation.

5.6.2 Uplift Resistance

The manholes should be designed to resist hydrostatic uplift. For uplift design of the underground structures, we recommend that the water level be assumed to be at the ground surface or 100-year flood elevation, whichever is more critical. If the dead weights of the structures are inadequate to resist uplift forces, toe extensions of the base slabs may be constructed so that the effective weight of the soil above the extended slabs can be utilized to resist the uplift forces. The unit buoyant weight of concrete can be taken as 90 pcf. The minimum recommended factors of safety against uplift should be 1.1 for concrete weight, 1.5 for soil weight and 3.0 for soil friction. Design soil parameters are included on Plate C-1 in Appendix C. Recommended design criteria for uplift resistance are shown on Plate D-8 in Appendix D.

5.6.3 Lateral Earth Pressures

There is no movement allowed for the walls of the manholes. Therefore, the walls should be designed for at-rest earth pressure. The magnitudes of these pressures will depend on the type and density of the backfill, surcharge on the backfill and hydrostatic pressure, if any. If the backfill is over-compacted or if highly plastic clays are placed behind the walls, the lateral earth pressure could exceed the vertical pressure. Typical backfill materials placed behind manhole walls in the Houston area include select fill and cement-stabilized sand.

Lateral pressure resulting from construction equipment or other surcharge should be taken into account by adding the equivalent uniformly distributed surcharge to the design lateral pressure. Hydrostatic pressure should also be included, unless adequate drainage is provided behind the walls. The at-rest earth pressure at depth z can be determined by the Equation (17) as follows:

$$p_0 = (q_s + \gamma\, h_1 + \gamma'\, h_2)\, K_0 + \gamma_w\, h_2 \qquad \text{........... Equation (17)}$$

Where, p_0 = at-rest earth pressure, psf;
q_s = uniform surcharge pressure, psf;

γ, γ’	’	wet and buoyant unit weights of soil, pcf, presented on Plate C-1 in Appendix C;
h_1	=	depth from ground surface to ground water table, feet;
h_2	=	$z-h_1$, depth from ground water table to point under consideration, feet;
z	=	depth below ground surface, feet;
K_0	=	coefficient of at-rest earth pressure, presented on Plate C-1 in Appendix C; and
γ_w	=	unit weight of water, 62.4 pcf.

5.6.4 Backfill Material

According to Section 02317 of the 2009 COHSCS, the manholes below paved areas shall be encapsulated with cement stabilized sand, minimum one foot below base, minimum one foot around walls, and up to within 12 inches of pavement subgrade. Cement-stabilized sand should comply with Section 02321 and compacted in accordance with Item 3.10, Section 02317 of the 2009 COHSCS, respectively. In unpaved areas, manhole backfill should consist of select fill placed as specified in Item 3.11.C, Section 02317 of the 2009 COHSCS; select fill should be Class IV Lean Clay in accordance with Item 2.02.I, Section 02320 of the 2009 COHSCS, and compacted to a minimum 95% of the ASTM D-698 maximum dry unit weight and at a moisture content between optimum and 3% wet of optimum.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 Site Preparation

To mitigate construction problems which may develop if attempts are made to work the surface materials following prolonged periods of rainfall, we recommend that prior to starting any work at the site, proper construction drainage be provided to maintain a relatively dry construction site. Improper site drainage can result in wet surface soil conditions and excessive disturbance to the subgrade soils. Methods for controlling surface runoff and ponding include proper site grading, berm construction around exposed areas, and installation of sump pits with pumps etc.

6.2 Groundwater Control

According to our water level reading in Piezometers PZ-1 and PZ-2, the long term ground water level is at depths ranging from about 5 to 10 feet below existing ground surface. However, the groundwater conditions might vary during construction. In the event that there is heavy rain prior to or during construction, the groundwater table may be higher than that indicated in this report; higher seepage is also likely and may require a more extensive groundwater control program. In addition, groundwater may be pressurized in certain areas of the alignment, requiring further evaluation and consideration of the excess hydrostatic pressures.

The Contractor should be responsible for selecting, designing, constructing, maintaining and monitoring a groundwater control system and adapt his augering operations to ensure the stability of the excavations. Groundwater information presented in Section 4.4 and elsewhere in this report, along with consideration for potential environmental and site variation between the time of our field exploration and construction, should be incorporated in evaluating groundwater depths.

We recommend that the Contractor verify the groundwater depths and seepage rates and existence of pressurized groundwater prior to and during construction and retain the services of a dewatering expert to assist him in identifying the most suitable and cost-effective method of controlling groundwater, if needed. The Contractor should take necessary precautions to avoid distressing existing structures as a result of dewatering. The following recommendations are intended to guide the Contractor during design and construction of the dewatering system. In saturated deposits of granular soils, groundwater is typically controlled by the installation of vacuum well points. Close well point spacing is required if the granular soils are fine-grained; this spacing is typically on the order of 10 to 20 feet on center. The practical maximum depth for the use of vacuum well points is considered to be around 15 feet. When groundwater control is required below 15 feet, deep wells with submersible pumps have generally proved successful.

In cohesive soils seepage rates are lower than in granular soils and groundwater is usually collected in sumps and channeled by gravity flow to storm sewers. If cohesive soils contain significant secondary features, seepage rates will be higher. This may require larger sumps and drainage channels, or if significant granular layers are interbedded within the cohesive soils, methods used for granular soils may be required. Where it is present, pressurized groundwater will also yield higher seepage rates.

6.3 Construction Monitoring

Site preparation (including clearing and proof-rolling), earthwork operations, and foundation construction should be monitored by qualified geotechnical professionals to check for compliance with project documents and changed conditions, if encountered. We recommend that AEC be allowed to monitor the construction and installation of pavement, storm/sanitary sewers, water lines and other items related to our investigation.

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7.0 GENERAL

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the dates of drilling. Reasonable variations from the subsurface information presented in this report should be anticipated. AEC should be allowed to review the design and construction plans and specifications prior to release to check that the geotechnical recommendations and design criteria presented herein are properly interpreted. If conditions encountered during construction are significantly different from those presented in this report; AEC should be notified immediately.

8.0 LIMITATIONS

This investigation was performed using the standard level of care and diligence normally practiced by recognized geotechnical engineering firms in this area, presently performing similar services under similar circumstances. This report is intended to be used in its entirety. The report has been prepared exclusively for the project and location described in this report. If pertinent project details change or otherwise differ from those described herein, AEC should be notified immediately and retained to evaluate the effect of the changes on the recommendations presented in this report, and revise the recommendations if necessary. The recommendations presented in this report should not be used for other structures located along this alignment or similar structures located elsewhere, without additional evaluation and/or investigation.

9.0 CLOSING REMARKS

AEC appreciates the opportunity to be of service on this project and looks forward to our continuing association during the construction phase of this project and on future projects.

AVILES ENGINEERING CORPORATION
(TBPE Firm Registration No.: 42)

STATE OF TEXAS
PENG SIA TANG
81150
REGISTERED
PROFESSIONAL ENGINEER
6/6/11

Peng Sia Tang , M.S.C.E., P.E.
Senior Engineer

Chun Ho Lee, M.S.C.E., E.I.T.
Graduate Engineer

June 6, 2011

Copies Submitted:	Omega Engineers, Inc.	3	Plus 1 PDF
	File	1	

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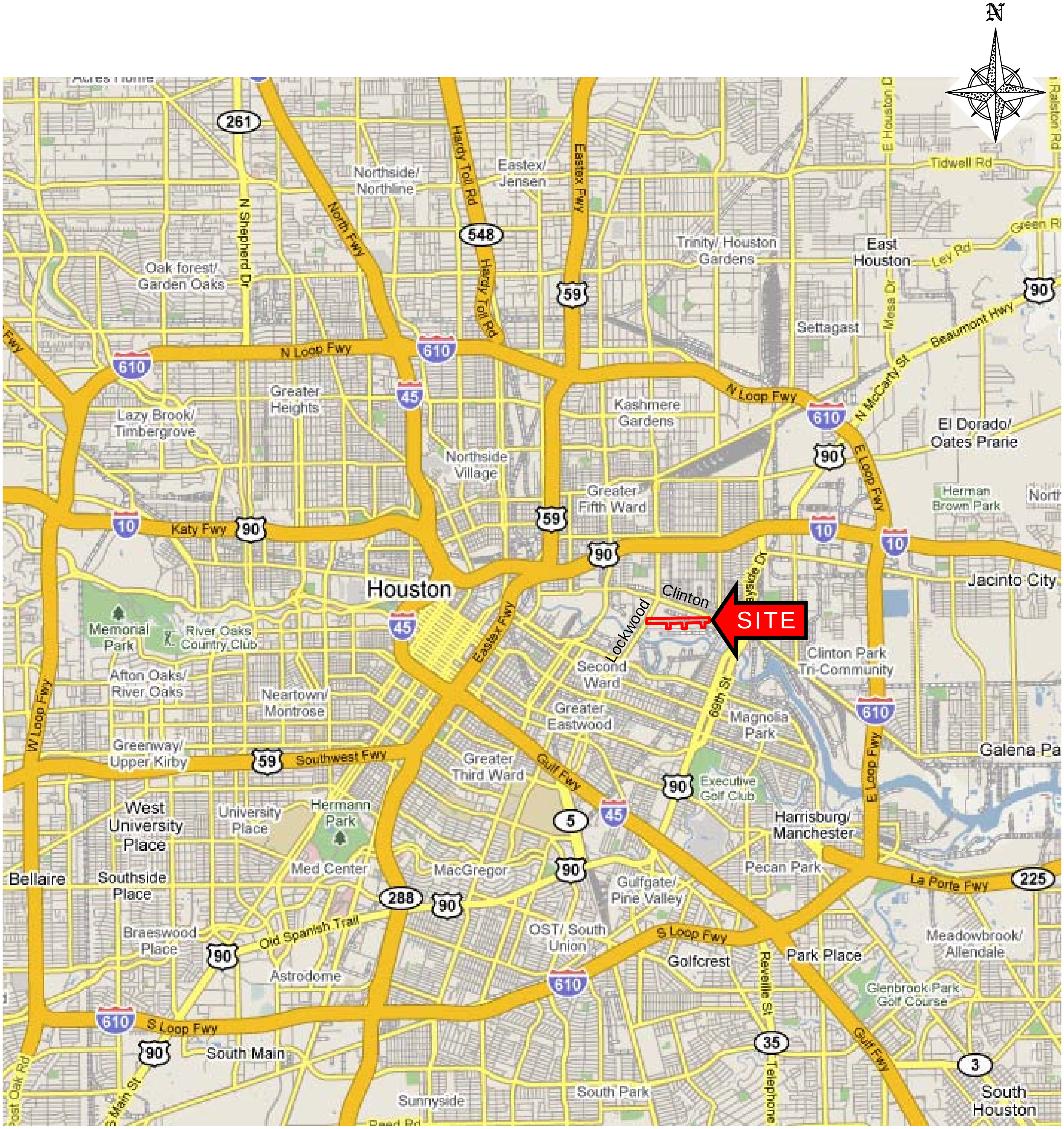
APPENDIX A

Plate A-1	Vicinity Map
Plates A-2a to A-2c	Boring Location Plan
Plates A-3 to A-16	Boring Logs
Plate A-17	Key to Symbols
Plate A-18	Classification of Soils for Engineering Purposes
Plate A-19	Terms Used on Boring Logs
Plate A-20	ASTM & TXDOT Designation for Soil Laboratory Tests

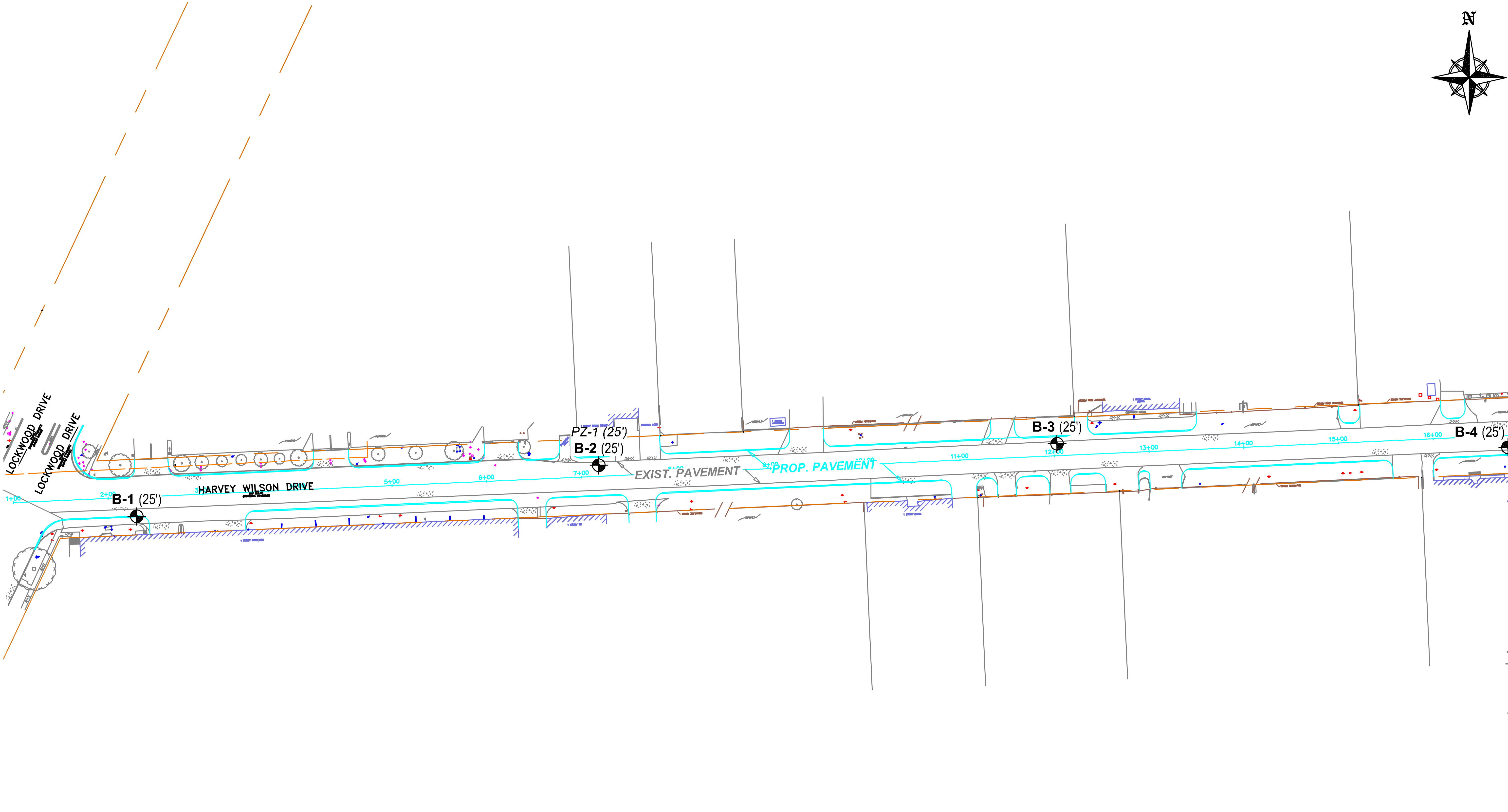


AVILES ENGINEERING CORPORATION

VICINITY MAP

HARVEY WILSON ROAD RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	
G169-09	10-19-09	
APPROX. SCALE: N.T.S.	DRAFTED BY: BpJ	PLATE NO.: PLATE A-1



AVILES ENGINEERING CORPORATION

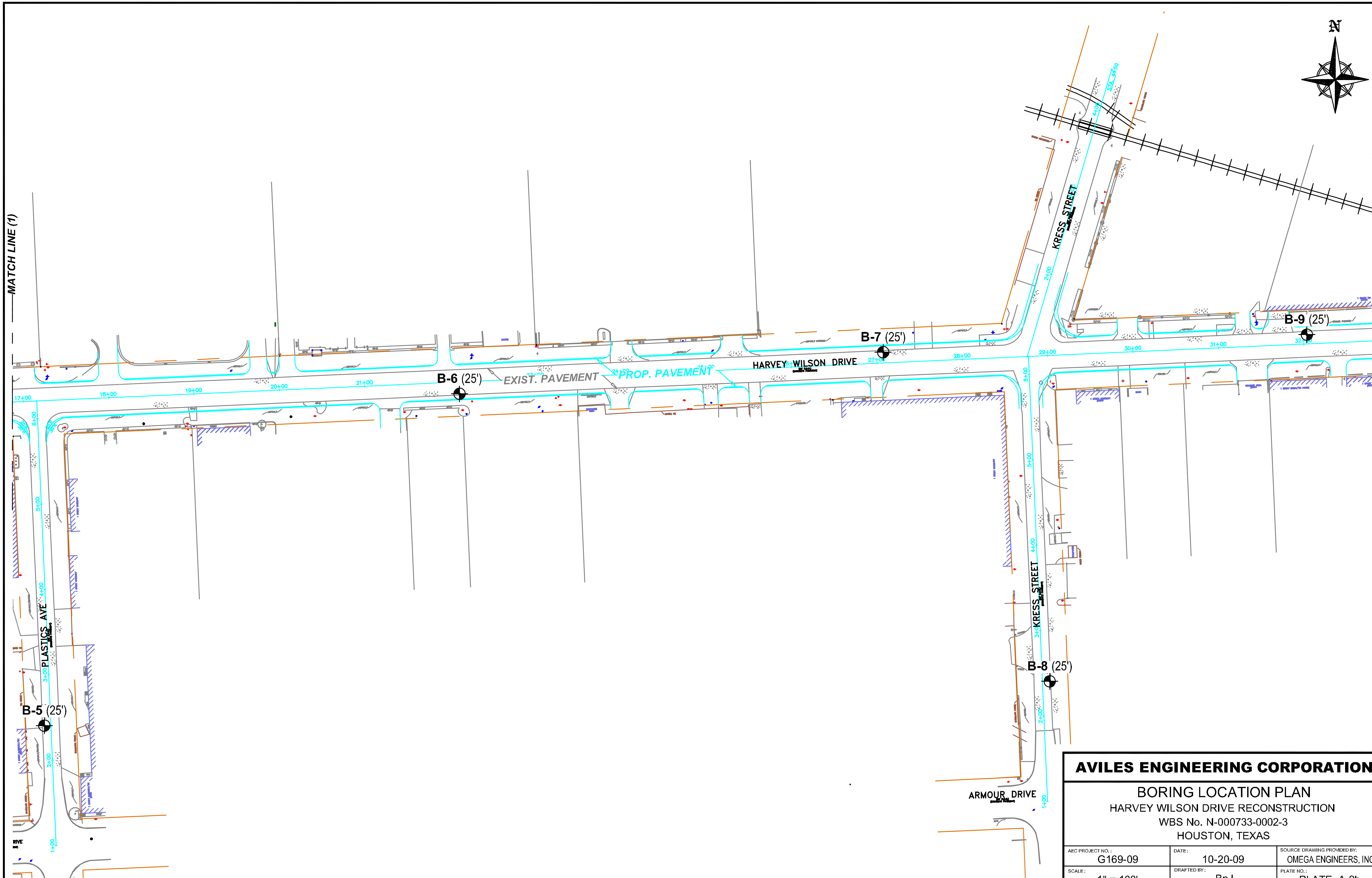
BORING LOCATION PLAN

HARVEY WILSON DRIVE RECONSTRUCTION

WBS No. N-000733-0002-3

HOUSTON, TEXAS

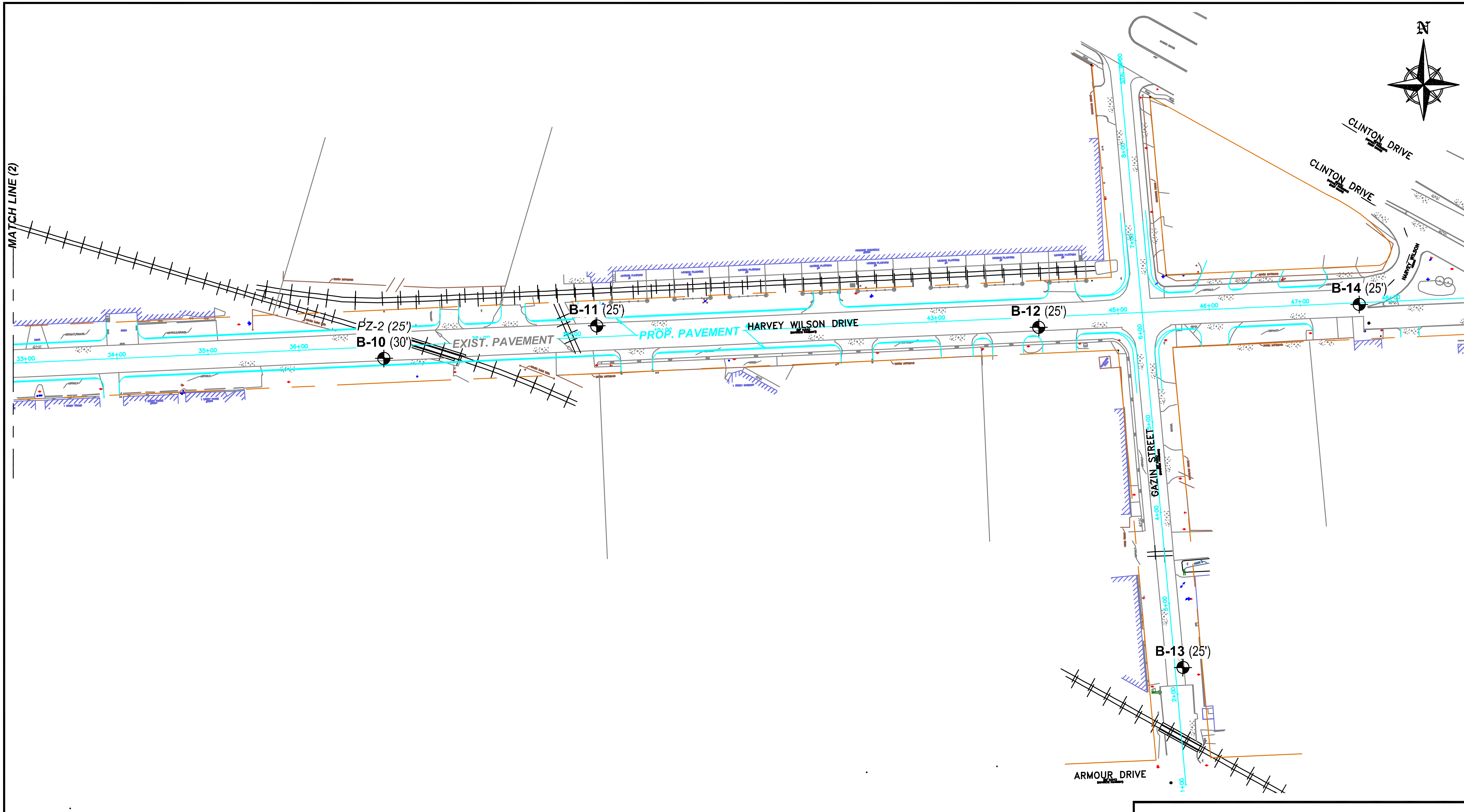
AEC PROJECT NO.: G169-09	DATE: 10-20-09	SOURCE DRAWING PROVIDED BY: OMEGA ENGINEERS, INC.
SCALE: 1" = 100'	DRAFTED BY: BpJ	PLATE NO.: PLATE A-2a



AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN
HARVEY WILSON DRIVE RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.: G169-09	DATE: 10-20-09	SOURCE DRAWING PROVIDED BY: OMEGA ENGINEERS, INC.
SCALE: 1" = 100'	DRAFTED BY: BpJ	PLATE NO.: PLATE A-2b



AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN
HARVEY WILSON DRIVE RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.: G169-09	DATE: 10-20-09	SOURCE DRAWING PROVIDED BY: OMEGA ENGINEERS, INC.
SCALE: 1" = 100'	DRAFTED BY: BpJ	PLATE NO.: PLATE A-2c

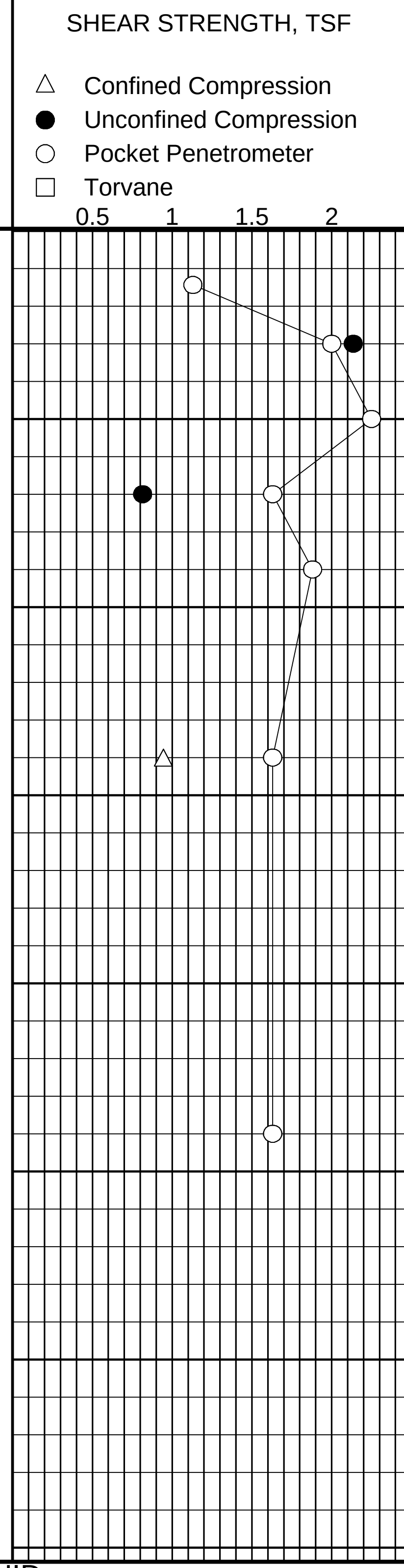
PROJECT: **Harvey Wilson Drive Reconstruction**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-1**

DATE **8/25/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

Depth (ft)	Description	S.P.T. Blows / Ft.	Moisture Content, %	Dry Density, PCF	-200 Mesh	Liquid Limit	Plastic Limit	Plasticity Index
0	Pavement: 1.6" ACC and 8.9" PCC							
	Very stiff, dark gray Fat Clay (CH), with 3" sand layer at surface, and ferrous nodules		27			57	14	43
	Tan and light gray Clayey Sand (SC), with calcareous and ferrous nodules		17	111				
5			14		48	33	11	22
			14	113		29	12	17
			16		38			
10								
	Stiff to very stiff, reddish brown and light gray Fat Clay (CH), with slickensides, calcareous nodules, and siltstone fragments							
			25	102				
15								
	Dense, tan and light gray Clayey Sand (SC), with sand seams					28	11	17
		33	19					
20	Very stiff, light gray and brown Fat Clay (CH), with calcareous nodules							
			24		93			
25	Termination depth = 25 feet.							
30								
35								



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **12.3** FEET WHILE DRILLING

WATER LEVEL AT **10.6** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

PLATE A-3

PROJECT: **Harvey Wilson Drive Reconstruction**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

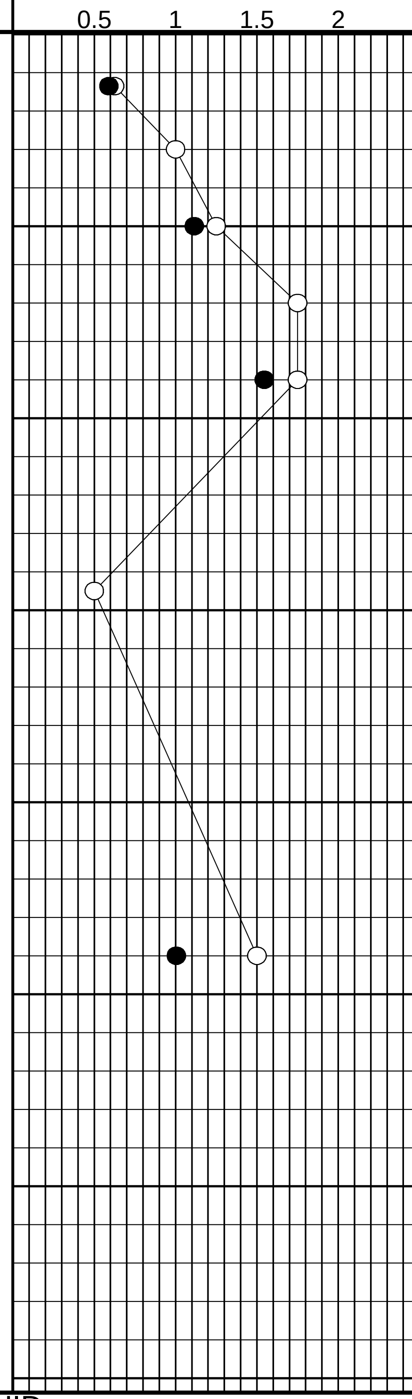
BORING **B-2**

DATE **8/25/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 1.4" ACC and 7.1" PCC							
	Fill: stiff, dark gray Fat Clay (CH), with sandy clay seams and ferrous stains		24	99				
	Stiff to very stiff, gray and tan Sandy Lean Clay (CL), with roots 2'-6', and ferrous stains					47	12	35
			21					
5	-light gray and tan 4'-10', with calcareous nodules 4'-6'		15	112				
	-with sand pockets 6'-8'		19					
			18	112		35	11	24
10								
15	Medium dense, tan and light gray Clayey Sand (SC), with abundant ferrous stains		20		33			
		12						
20	Stiff, tan and light gray Sandy Lean Clay (CL), with silt partings and ferrous stains		19		59			
	Very stiff, tan and light gray Fat Clay (CH), with slickensides		25	100		65	15	50
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **14.4** FEET WHILE DRILLING

WATER LEVEL AT **10.5** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

PLATE A-4

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Harvey Wilson Drive Reconstruction**

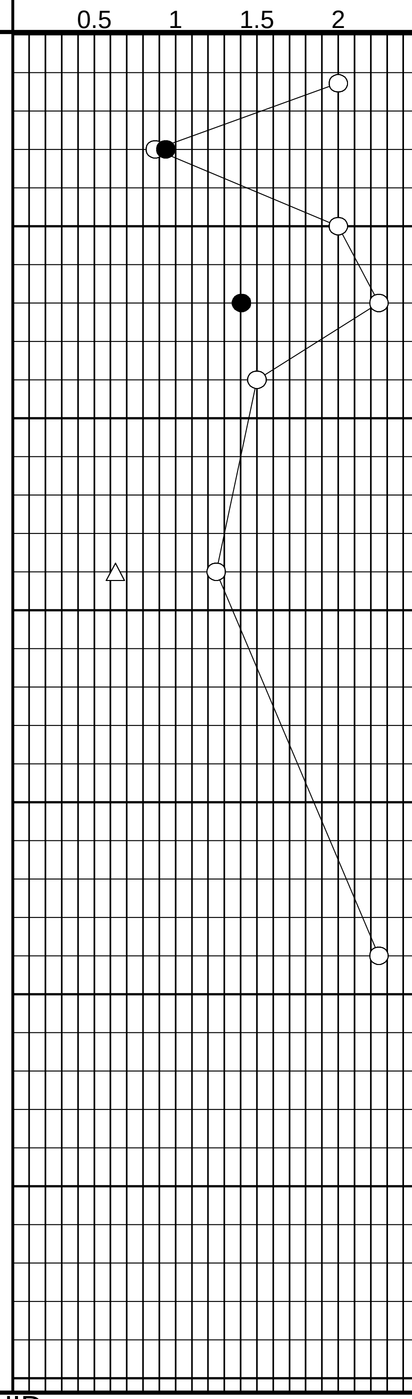
BORING **B-3**

DATE **8/25/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Pavement: 6.75" PCC							
Very stiff, tan and light gray Lean Clay w/ Sand (CL), with ferrous stains		13		74			
Stiff to hard, dark gray Sandy Lean Clay (CL), with ferrous stains		19	105				
-gray and tan 4'-6'					34	12	22
		15					
-light gray and tan 6'-15', with sand pockets 6'-8'		15	113				
				63	30	12	18
		16					
-with sand pockets 13'-15'		17	107				
Very stiff, light gray and red Lean Clay (CL), with silt partings					37	15	22
	17	25					
Hard, light gray and tan Fat Clay (CH), with calcareous nodules and ferrous stains		20					
Termination depth = 25 feet.							

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



DEPTH IN FEET: 0, 5, 10, 15, 20, 25, 30, 35

BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **13.3** FEET WHILE DRILLING
WATER LEVEL AT **11.0** FEET AFTER **15 MIN.**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT: **Harvey Wilson Drive Reconstruction**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

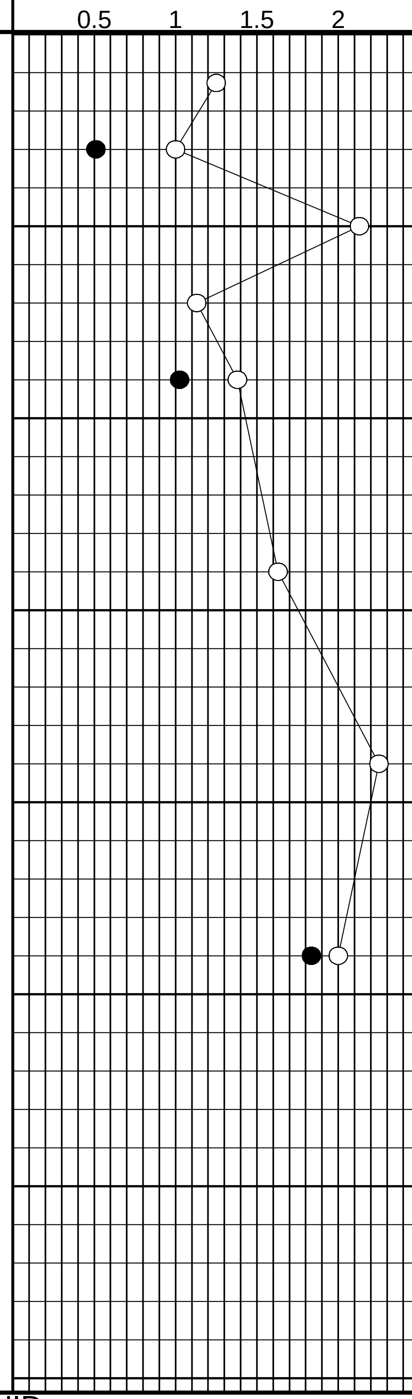
BORING **B-4**

DATE **8/25/09** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 6.5" PCC							
	Fill: stiff to hard, dark gray and light gray Sandy Fat Clay (CH), with calcareous nodules and ferrous stains		21			61	19	42
	-with sand pockets 0'-2'		19	99				
5	-light gray and tan 2'-4', with wood pieces 2'-8'		15			51	14	37
	-gray and tan, with sand partings 4'-6'				54			
			17					
	Very stiff, gray and tan Lean Clay w/Sand (CL), with ferrous stains		17	112		34	12	22
10								
			20		84			
15	-borehole caved in at 15.3'							
	Very stiff to hard, tan and light gray Fat Clay (CH), with ferrous stains		22			58	18	40
20								
			21	104				
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **20.3** FEET WHILE DRILLING

WATER LEVEL AT **15.3** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT: **Harvey Wilson Drive Reconstruction**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

BORING **B-5**

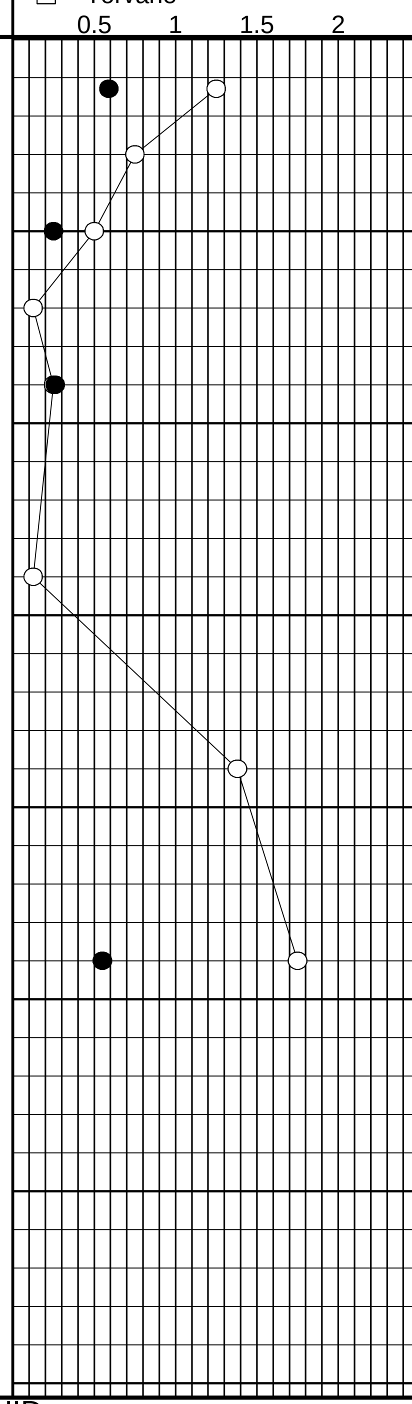
DATE **8/25/09** TYPE **4" Dry Auger** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION
0	Pavement: 1" AC and 6" PCC
	Fill: stiff to very stiff, dark gray Sandy Lean Clay (CL), with slickensides, sand partings and ferrous stains
	Fill: stiff, gray Sandy Silty Clay (CL-ML), with sand seams
5	Fill: very soft to firm, gray Sandy Lean Clay (CL), with sand pockets and roots 4'-6'
	-gray and tan 8'-15', with sand pockets 8'-10'
18	Stiff to very stiff, reddish brown and light gray Fat Clay (CH), with slickensides
	-with calcareous nodules 18'-20'
	-tan and light gray 23'-25'
25	Termination depth = 25 feet.

S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
				39	12	27
	16	107				
	17		56	21	14	7
	27	91				
	24			39	12	27
	31	84				
	30					
	29			67	20	47
	22	102				

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **25** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **21** FEET WHILE DRILLING
WATER LEVEL AT **15.3** FEET AFTER **15 MIN.**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

PLATE A-7

PROJECT: **Harvey Wilson Drive Reconstruction**

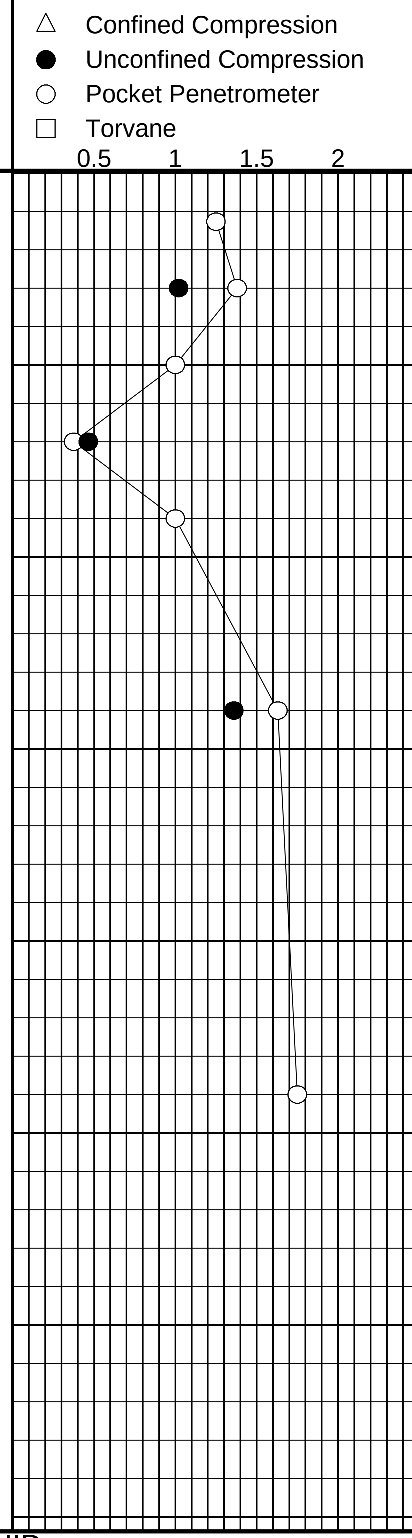
AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-6**

DATE **8/25/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 6.5" PCC							
	Very stiff, gray and tan Fat Clay (CH), with silt seams and partings -with calcareous nodules 2'-4'					60	19	41
			18					
			20	106				
5	Firm to very stiff, tan and gray Sandy Lean Clay (CL), with sand seams 4'-6', and ferrous stains -tan and light gray 6'-15', with silt partings 6'-8'					31	12	19
			16					
					70			
			17	110				
			15					
10								
						34	13	21
			15	113				
15	Silty Sand (SM)							
	Very stiff, reddish brown Fat Clay (CH), with sand seams -sand layer 19.5'-20'				57			
		21	37					
20								
	-tan and light gray, with sand pockets 23'-25'					55	15	40
			21					
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **9** FEET WHILE DRILLING

WATER LEVEL AT **8** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

PLATE A-8

PROJECT: **Harvey Wilson Drive Reconstruction**

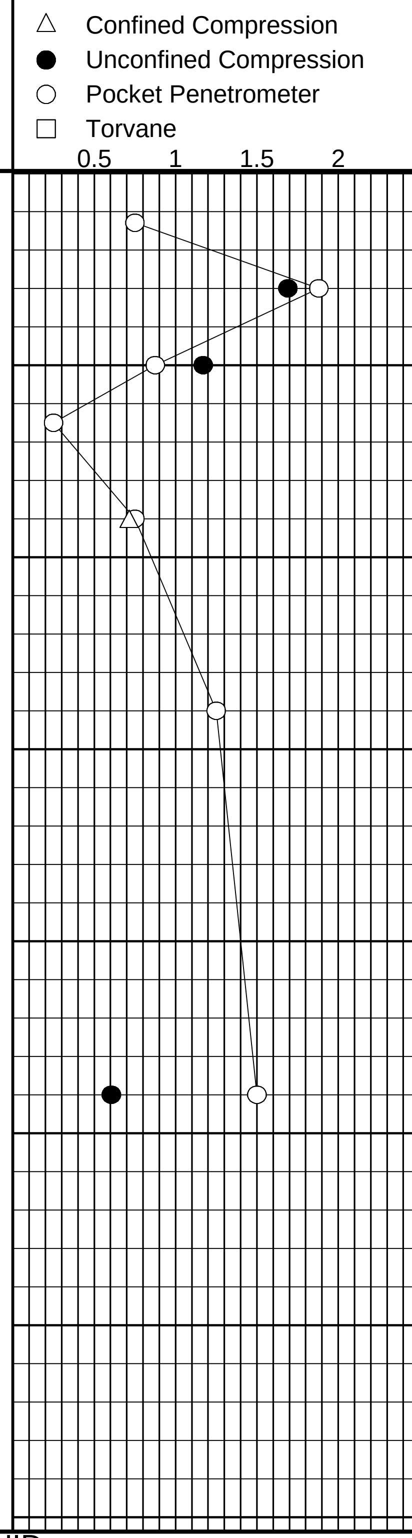
AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-7**

DATE **8/25/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 7" PCC							
	Fill: stiff, gray and tan Sandy Lean Clay (CL), with sand pockets		15			30	12	18
	Stiff to very stiff, gray and tan Lean Clay w/ Sand (CL), with sand pockets 2'-4'		15	114	73			
5	-with calcareous and ferrous nodules 4'-6'		16	112				
	Firm, gray Silty Clay w/Sand (CL-ML)				74			
	Stiff to very stiff, light gray and tan Lean Clay w/Sand (CL), with silt partings and ferrous nodules 8'-10'		21			29	12	17
			22	106				
10								
	-with silt pockets 13'-15'		23		78	27	13	14
15	Medium dense, light gray Silty Sand (SM)				26			
		29	23					
20	Stiff to very stiff, light gray and tan Fat Clay (CH), with slickensides							
			24	101				
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **11** FEET WHILE DRILLING

WATER LEVEL AT **7.5** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

PLATE A-9

PROJECT: **Harvey Wilson Drive Reconstruction**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-8**

DATE **8/25/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 1.4" ACC and 7.1" PCC							
	Stiff to very stiff, gray and tan Sandy Lean Clay (CL), with ferrous stains		19	107		37	12	25
					52			
			16					
						29	12	17
5			15	112				
	Medium dense, tan and gray Clayey Sand (SC), with sand pockets 6'-8'				42			
			17					
						34	12	22
		19	16					
10	Stiff to very stiff, light gray and tan Sandy Lean Clay (CL), with sand pockets							
			22	103				
15	Stiff to very stiff, reddish brown and light gray Fat Clay (CH), with slickensides							
	-with calcareous nodules, siltstone fragments and silt pockets 18'-20'					60	19	41
			24					
20								
	-light gray and tan, with ferrous stains 23'-25'							
			24	100				
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane

0.5 1 1.5 2

BORING DRILLED TO **10** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **8** FEET WHILE DRILLING

WATER LEVEL AT **7.2** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

PLATE A-10

PROJECT: **Harvey Wilson Drive Reconstruction**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

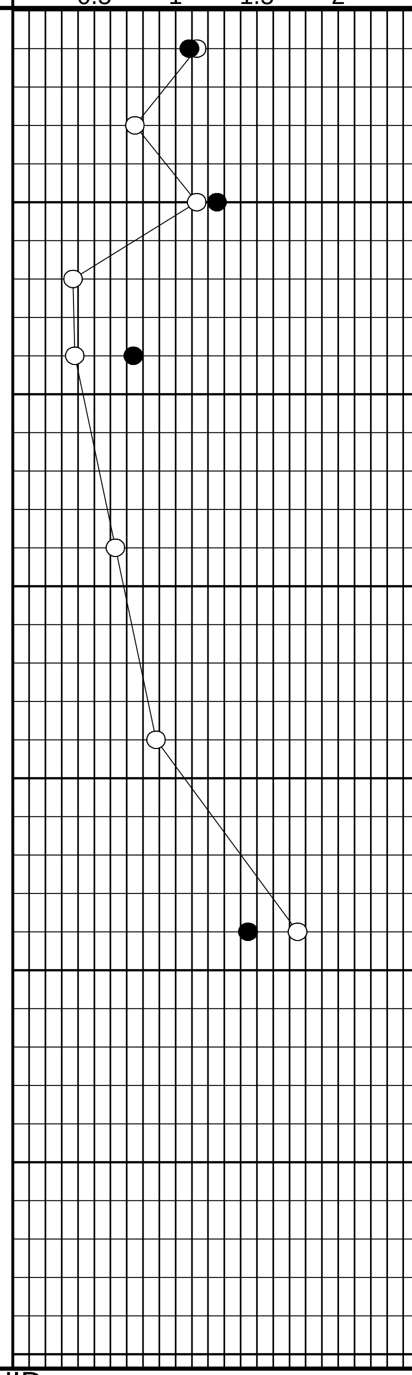
BORING **B-9**

DATE **8/26/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 7" PCC							
	Fill: very stiff, gray and tan Sandy Fat Clay (CH), with ferrous stains		24	104				
	Stiff to very stiff, gray and tan Sandy Fat Clay (CH), with ferrous stains and calcareous nodules					57	17	40
			22					
5			18	109				
	Firm to stiff, tan and light gray Lean Clay w/ Sand (CL), with ferrous stains and silt partings -with sand/silt lenses 6'-8'		15		75	29	14	15
			17	110				
10								
						38	13	25
			18					
15								
					83	28	18	10
			22					
20	Very stiff, light gray and tan Fat Clay (CH), with slickensides							
			21	104				
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **16.5** FEET WHILE DRILLING

WATER LEVEL AT **9.9** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

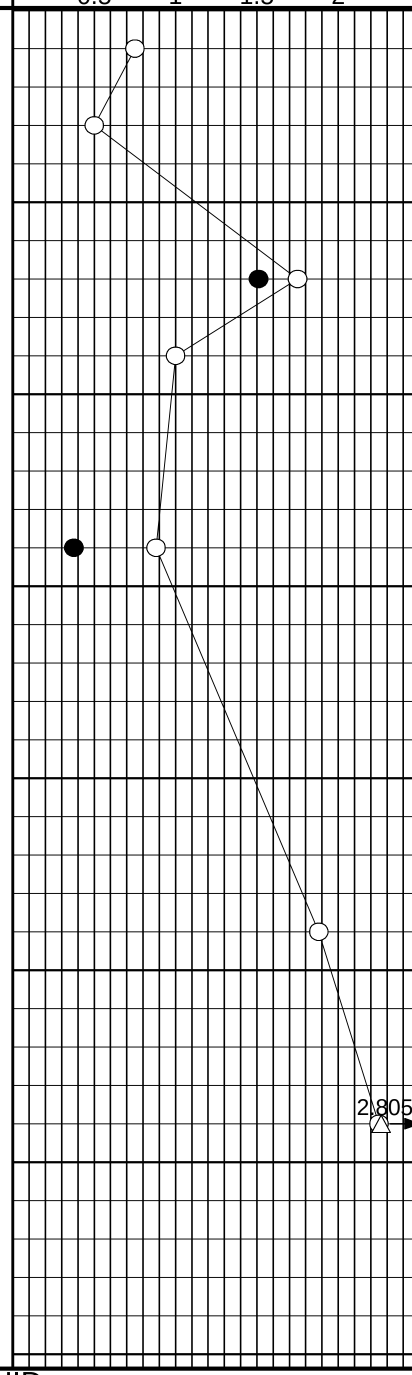
PROJECT: **Harvey Wilson Drive Reconstruction**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

BORING **B-10**

DATE **8/25/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

SHEAR STRENGTH, TSF: △ Confined Compression ● Unconfined Compression ○ Pocket Penetrometer □ Torvane (scale 0.5, 1, 1.5, 2)



DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 1.3" AC and 7.7" PCC							
	Fill: firm to stiff, light gray and tan Lean Clay w/Sand (CL), with silt seams -with sand seams and pockets 0'-2'		22		80			
						32	12	20
			18					
5	Firm to very stiff, light gray and tan Sandy Lean Clay (CL), with ferrous stains -with sand seams 4'-6'					45	11	34
		17	16					
			16	111				
						30	12	18
			17					
10								
					56			
			17	112				
15	Dense, light gray and tan Silty Sand (SM)							
					42			
		35	19					
20	Very stiff to hard, light gray and reddish tan Fat Clay (CH), with ferrous stains							
			37					
25								
	-with sand partings 28'-30'					54	15	39
			18	112				
30	Termination depth = 30 feet.							
35								

BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **9** FEET WHILE DRILLING

WATER LEVEL AT **6** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Harvey Wilson Drive Reconstruction**

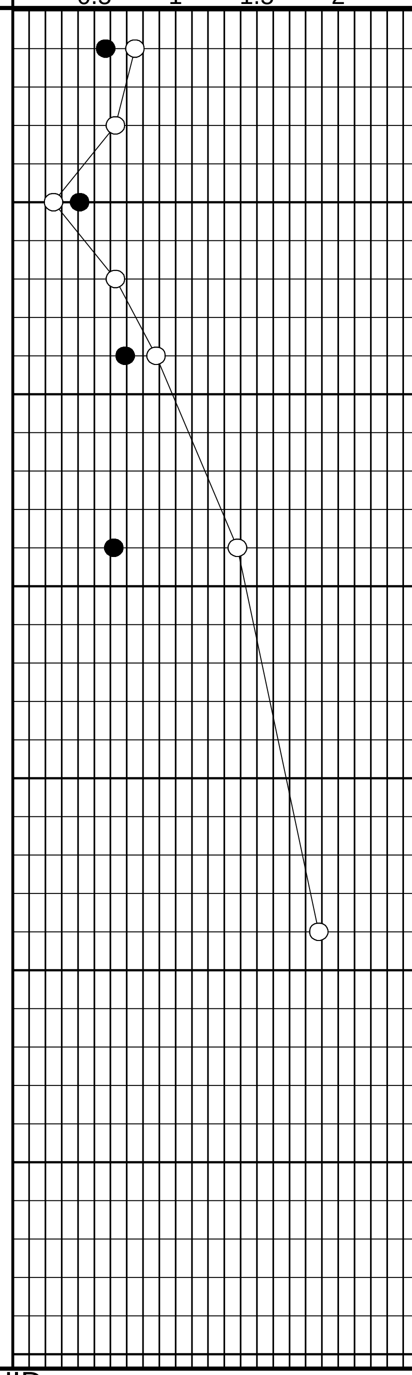
BORING **B-11**

DATE **8/26/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 0.85" AC and 5.65" PCC							
	Fill: stiff, gray and tan Sandy Lean Clay (CL), with sand pockets		16	106				
	Soft to very stiff, gray and tan Lean Clay w/ Sand (CL), with ferrous stains				77	35	12	23
	-with sand seams and pockets 2'-4'		19					
5	-with silt pockets 4'-6'		23	99				
	-with silt partings 6'-8'					37	12	25
			23					
	-with silt pockets 8'-10'		20	106				
10								
					75	40	13	27
			18	109				
15								
	Medium dense, light gray Silty Sand (SM)							
		29	22					
20	Very stiff, light gray and tan Sandy Fat Clay (CH), with ferrous stains and clay pockets					54	15	39
			19					
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF

△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
□ Torvane



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT **11.5** FEET WHILE DRILLING
WATER LEVEL AT **8.75** FEET AFTER **15 MIN.**
DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

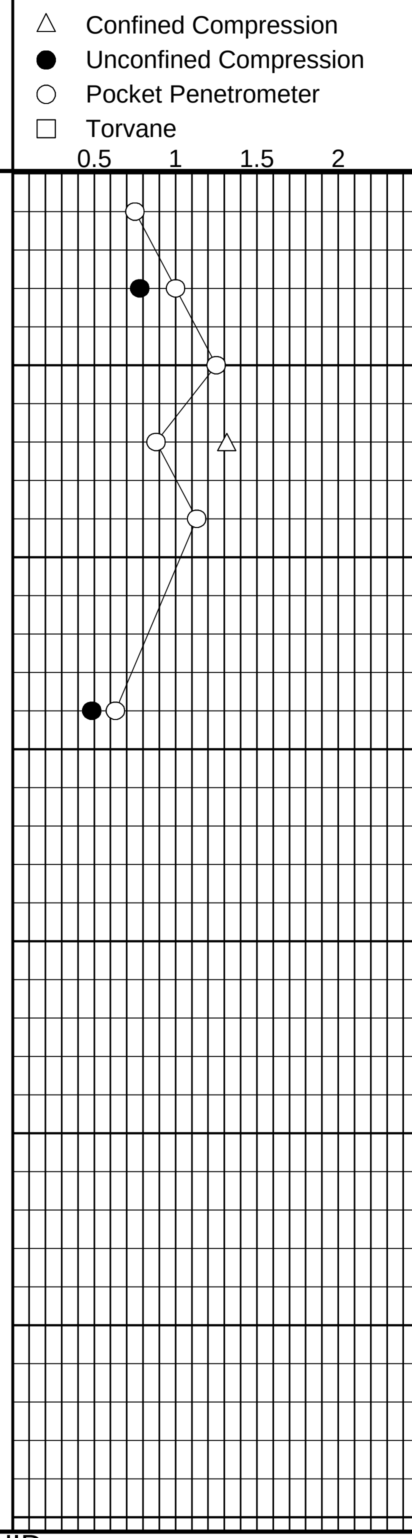
AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: **Harvey Wilson Drive Reconstruction** BORING **B-12**

DATE **8/26/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 8" PCC							
	Fill: stiff, gray and tan Fat Clay w/Sand (CH), with ferrous stains		19		81	51	13	38
	-with calcareous nodules and siltstone fragements 0'-2'		21	98				
	-with sand seams 2'-4'					48	12	36
5	Firm to very stiff, gray and tan Lean Clay w/ Sand (CL), with ferrous stains		22					
	-with siltstone fragments 6'-8'		17	111				
					82	28	15	13
			17					
10								
	-with sand seams 13'-15'		17	110				
15	Loose to medium dense, light gray Silty Sand (SM)							
	-with fat clay layers 20'-23'				40			
20		4	20					
		24	24					
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **12.6** FEET WHILE DRILLING

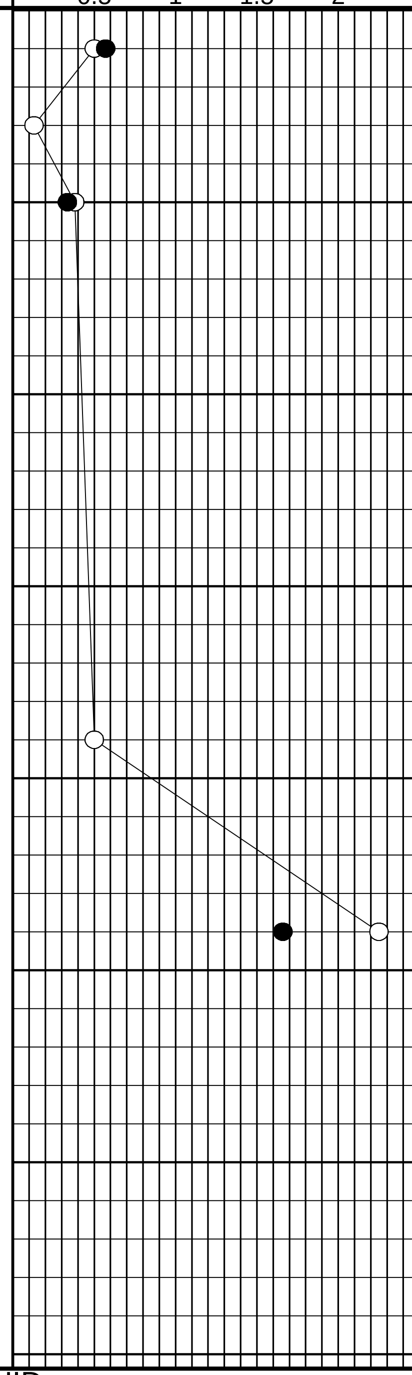
WATER LEVEL AT **9.9** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT: **Harvey Wilson Drive Reconstruction** | **AVILES ENGINEERING CORP.** GEOTECHNICAL ENGINEERS | BORING **B-13**

DATE **8/26/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

SHEAR STRENGTH, TSF: △ Confined Compression; ● Unconfined Compression; ○ Pocket Penetrometer; □ Torvane (scale 0.5, 1, 1.5, 2)



DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 7" PCC							
	Fill: soft to stiff, gray and tan Sandy Lean Clay (CL), with ferrous stains		25	97		37	12	25
	-with roots and sand seams 2'-4'		30			43	12	31
5	Very loose to loose, tan Silty Sand (SM) -with clay partings 4'-6'		20	103				
		4	20		37			
		9	20					
10	-boring caved in at 10'							
	Very soft, tan Sandy Lean Clay (CL), with silt partings	2	22		57	24	11	13
15								
	-gray and tan 18'-19'		22		15	38	12	26
20	Light gray and tan Silty Sand (SM)							
	Very stiff to hard, light gray and tan Sandy Fat Clay (CH)		16	113				
25	Termination depth = 25 feet.							
30								
35								

BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **9** FEET WHILE DRILLING

WATER LEVEL AT **N/A** FEET AFTER **N/A**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09 **PLATE A-15**

PROJECT: **Harvey Wilson Drive Reconstruction**

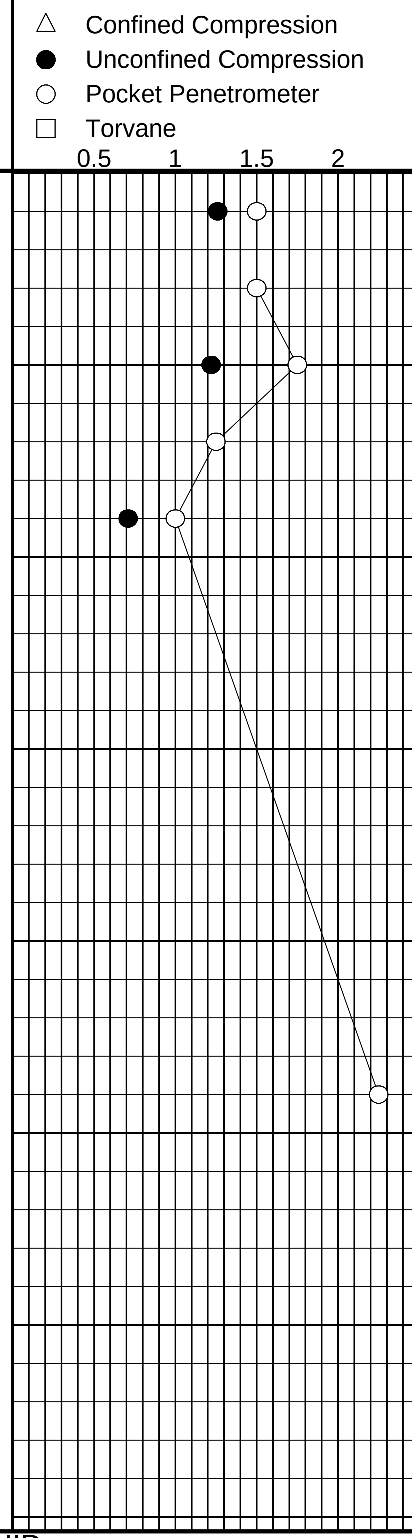
AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING **B-14**

DATE **8/26/09** TYPE **4" Dry Auger / Wet Rotary** LOCATION **See Boring Location Plan**

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 8" PCC							
	Very stiff, tan and gray Fat Clay (CH), with calcareous nodules and ferrous stains		20	108		51	13	38
	Stiff to very stiff, tan and gray Sandy Lean Clay (CL), with slickensides, calcareous nodules, ferrous stains and siltstone fragments		21					
5			21	108		36	14	22
			22					
	-with sand seams 8'-10'					29	13	16
			19	104				
10								
	Medium dense, tan and gray Silty Sand (SM)							
		11	22					
15								
	-with clayey sand pockets 18'-20'	24	26		32			
20								
	Hard, light gray and tan Lean Clay w/Sand (CL), with calcareous nodules, ferrous stains, siltstone fragments and sand pockets		20		85	31	11	20
25	Termination depth = 25 feet.							
30								
35								

SHEAR STRENGTH, TSF



BORING DRILLED TO **15** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **11** FEET WHILE DRILLING

WATER LEVEL AT **9.3** FEET AFTER **15 MIN.**

DRILLED BY **V&S** CHECKED BY **CHL** LOGGED BY **BOB METZGER**

PROJECT NO. G169-09

PLATE A-16

KEY TO SYMBOLS

Symbol	Description
	Strata symbols
	Paving
	High plasticity clay
	Clayey sand
	Fill
	Low plasticity clay
	Silty sand
	Silty low plasticity clay
	Description not given for: "CC"
	Soil Samplers
	Rock/pavement core
	Shelby Tube sampler

Symbol	Description
	Standard penetration test
	Misc Symbols
	Groundwater encountered during drilling
	Groundwater measured after drilling
○	Shear strength; pocket penetrometer
●	Shear strength; unconfined compression
△	Shear strength; confined compression

AVILES ENGINEERING CORP.

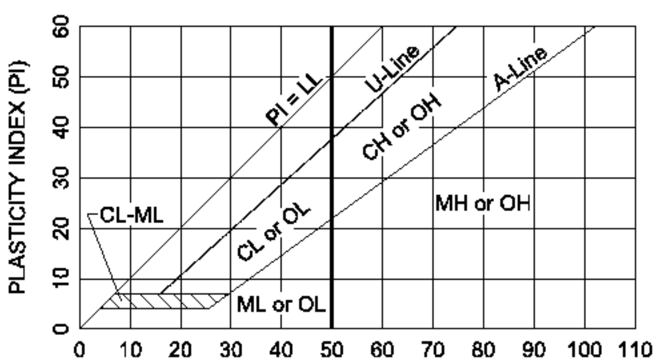
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (Less than 50% of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	Well-graded gravel, well-graded gravel with sand
				GP	Poorly-graded gravel, poorly-graded gravel with sand
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	GM	Silty gravel, silty gravel with sand
			Limits plot above "A" line & hatched zone on plasticity chart	GC	Clayey gravel, clayey gravel with sand
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	Well-graded sand, well-graded sand with gravel
				SP	Poorly-graded sand, poorly-graded sand with gravel
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	SM	Silty sand, silty sand with gravel
			Limits plot above "A" line & hatched zone on plasticity chart	SC	Clayey sand, clayey sand with gravel
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)		SILTS AND CLAYS (Liquid Limit Less Than 50%)		ML	Silt, silt with sand, silt with gravel, sandy silt, gravelly silt
				CL	Lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, gravelly lean clay
				OL	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt
		SILTS AND CLAYS (Liquid Limit 50% or More)		MH	Elastic silt, elastic silt with sand, sandy elastic silt, gravelly elastic silt
				CH	Fat clay, fat clay with sand, fat clay with gravel, sandy fat clay, gravelly fat clay
				OH	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt

NOTE: Coarse soils between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone of the plasticity chart are to have dual symbols.

PLASTICITY CHART



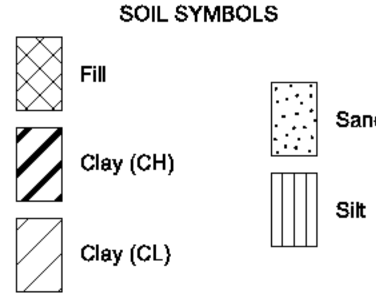
Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

Equation of U-Line: Vertical at LL=16 to PI=7, then PI=0.9(LL-8)

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None	0 - 4
Slight	5 - 10
Medium	11 - 20
High	21 - 40
Very High	>40

SOIL SYMBOLS



AVILES ENGINEERING CORP.

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

6"	3"	3/4"	#4	#10	#40	#200		
BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		
152	76.2	19.1	4.76	2.00	0.420	0.074	0.002	

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.
Very Soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very Stiff	2.00 to 4.00
Hard	greater than 4.00

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

Very Loose	<4 bpf
Loose	5-10 bpf
Medium Dense	11-30 bpf
Dense	31-50 bpf
Very Dense	>50 bpf

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows per Foot	Description
25	25 blows driving sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows driving sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows driving sampler 3 inches, during initial 6-inches seating interval.

NOTE: To avoid change to sampling tools, driving is limited to 50 blows during or after seating interval.

DRY STRENGTH ASTM D2488

None	Dry specimen crumbles into powder with mere pressure of handling
Low	Dry specimen crumbles into powder with some finger pressure
Medium	Dry specimen breaks into pieces or crumbles with considerable pressure
High	Dry specimen cannot be broken with finger pressure, it can be broken between thumb and hard surface
Very High	Dry specimen cannot be broken between thumb and hard surface

MOISTURE CONDITION ASTM D2488

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil types.
Interlayered	Soil sample composed of alternating layers of different soil types.
Intermixed	Soil sample composed of pockets of different soil types and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of calcium material.

AVILES

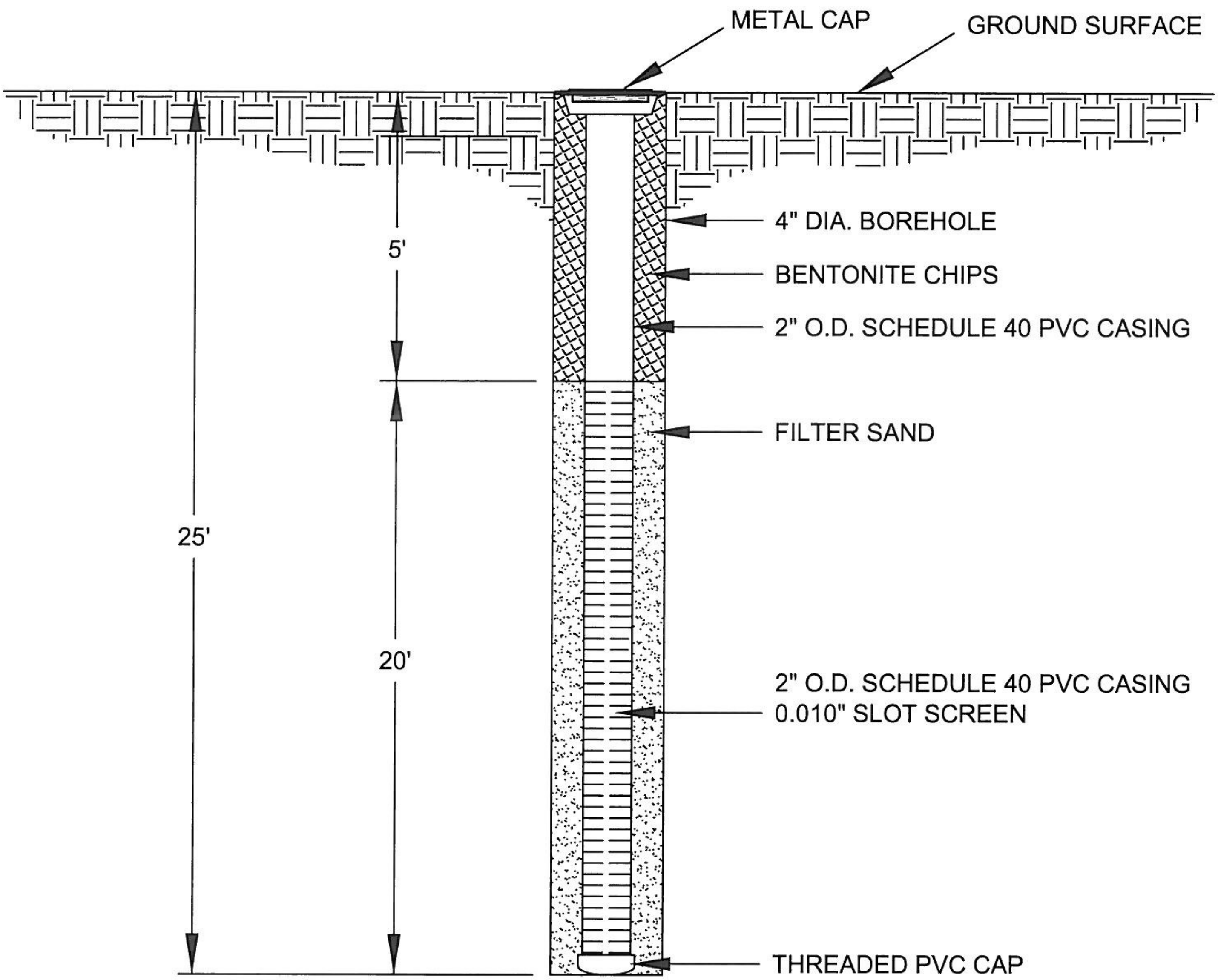
ASTM AND TxDOT DESIGNATIONS FOR LABORATORY SOIL TESTS

TEST	ASTM DESIGNATION	TxDOT DESIGNATION
Sieve Analysis	D 421 D 422	Tex-110-E (Part I)
Hydrometer Analysis	D 422	Tex-110-E (Part II)
Shrinkage Limit	D 427	Tex-107-E
Standard Proctor Compaction	D 698	Tex-114-E
Specific Gravity	D 854	Tex-108-E
Minus No. 200 Sieve	D 1140	Tex-111-E
Moisture Content	D 2216	Tex-103-E
Modified Proctor Compaction	D 1557	Tex-113-E
California Bearing Ratio (CBR)	D 1883	-
Unconfined Compression	D 2166	-
Permeability (constant head)	D 2434	-
Consolidation	D 2435	-
Unified Classification System	D 2487	Tex-142-E
Triaxial - Unconsolidated Undrained	D 2850	Tex-118-E
Direct Shear	D 3080	-
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	Tex-105-E
CBR of Soils In Place	D 4429	-
Crumb	Army Corps of Engineers Eng. Manual E1110-2-1906	
Pinhole	D 4647	-
Triaxial - Consolidated Undrained	D 4767	Tex-131-E

Note: Consolidated-Undrained Triaxial test is performed under three confining/effective stresses using a single specimen.

APPENDIX B

Plates B-1 and B-2	Piezometer Installation Details
Plates B-3 to B-5	Generalized Soil Profiles



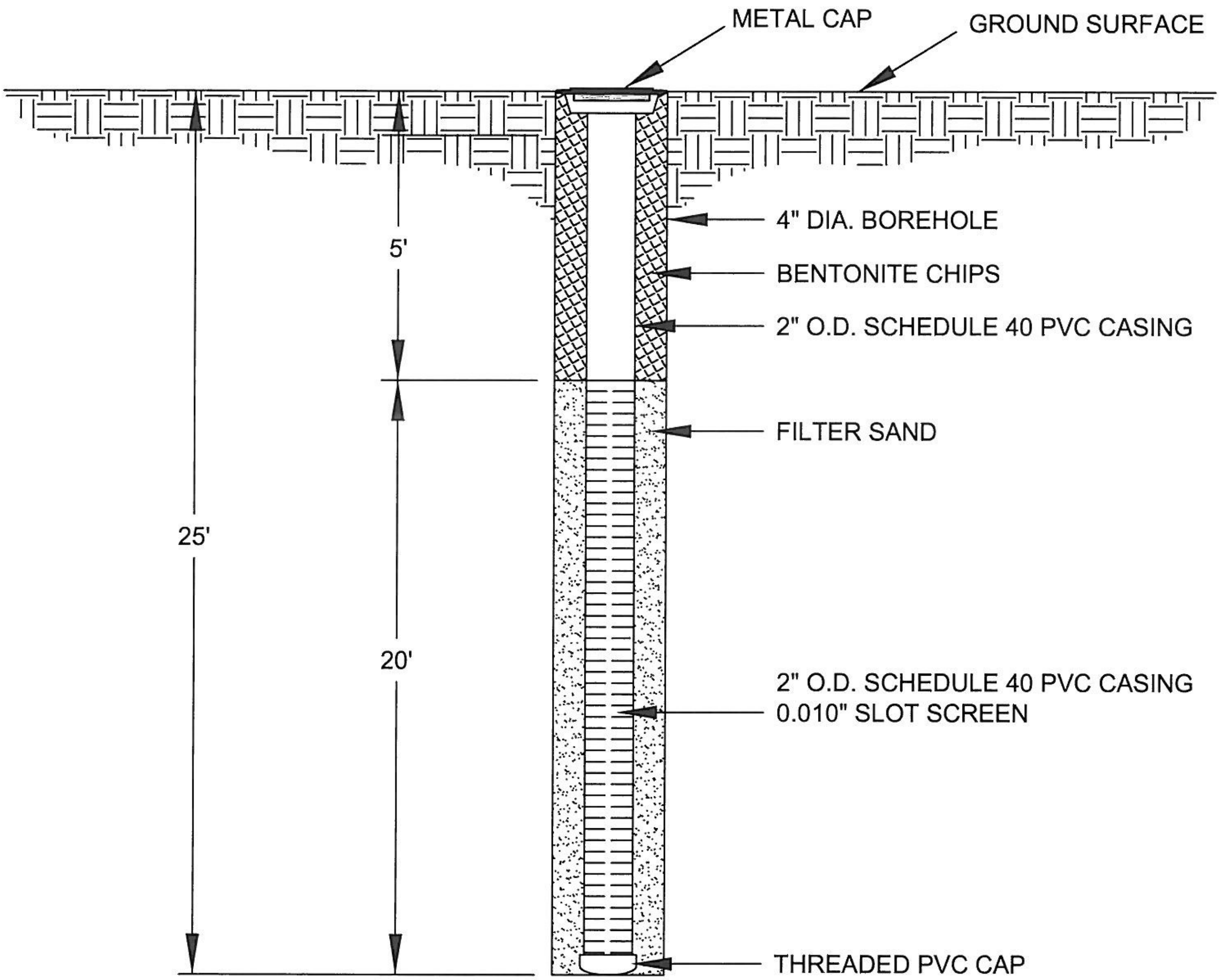
GROUNDWATER DEPTH FROM SURFACE:	DATE MEASURED:
9.7 ft.	8/28/09 (24 hr.)
9.6 ft.	9/28/09 (30 day)

AVILES ENGINEERING CORPORATION

PIEZOMETER INSTALLATION DETAILS
BORING B-2 (PZ-1)

HARVEY WILSON DRIVE RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.:	DATE:	SOURCE DWG. BY:
G169-09	10-19-09	AVILES ENGINEERING CORP.
SCALE:	DRAWN BY:	PLATE NO.:
N.T.S.	BpJ	PLATE B-1



GROUNDWATER DEPTH FROM SURFACE:	DATE MEASURED:
5.1 ft.	8/28/09 (24 hr.)
5.0 ft.	9/28/09 (30 day)

AVILES ENGINEERING CORPORATION

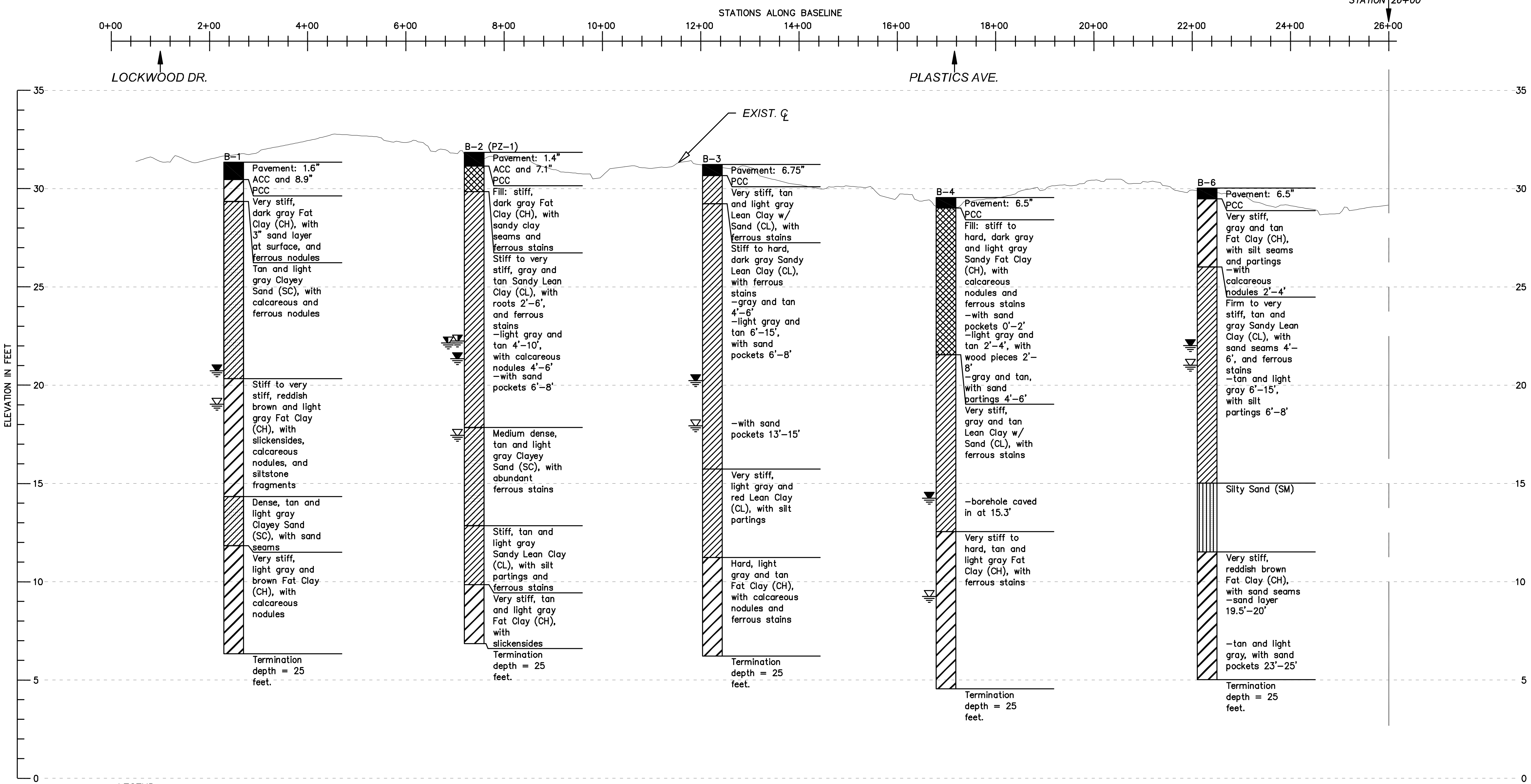
PIEZOMETER INSTALLATION DETAILS
BORING B-10 (PZ-2)

HARVEY WILSON DRIVE RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.: G169-09	DATE: 10-19-09	SOURCE DWG. BY: AVILES ENGINEERING CORP.
SCALE: N.T.S.	DRAWN BY: BpJ	PLATE NO.: PLATE B-2

GENERALIZED SUBSURFACE SOIL PROFILE

ALONG HARVEY WILSON DRIVE



LEGEND:

Pavement	Silty low plasticity clay
Fill	Silty sand
High plasticity clay	Clayey sand
Low plasticity clay	

- Depth of groundwater (or cave-in) encountered during drilling
- Depth of groundwater after approx. 15 minutes
- Depth of groundwater after approx. 24 hours
- Depth of groundwater after approx. 30 days

NOTE:
SOIL STRATIGRAPHY AND SECONDARY SOIL STRUCTURE (SUCH AS SEAMS, LAYERS, OR POCKETS OF SANDS, SILTS, SLICKENSIDES, AND FISSURES) THAT ARE DIFFERENT FROM WHAT WERE IDENTIFIED IN THE ACTUAL BORINGS MAY EXIST AWAY FROM THESE BORINGS.

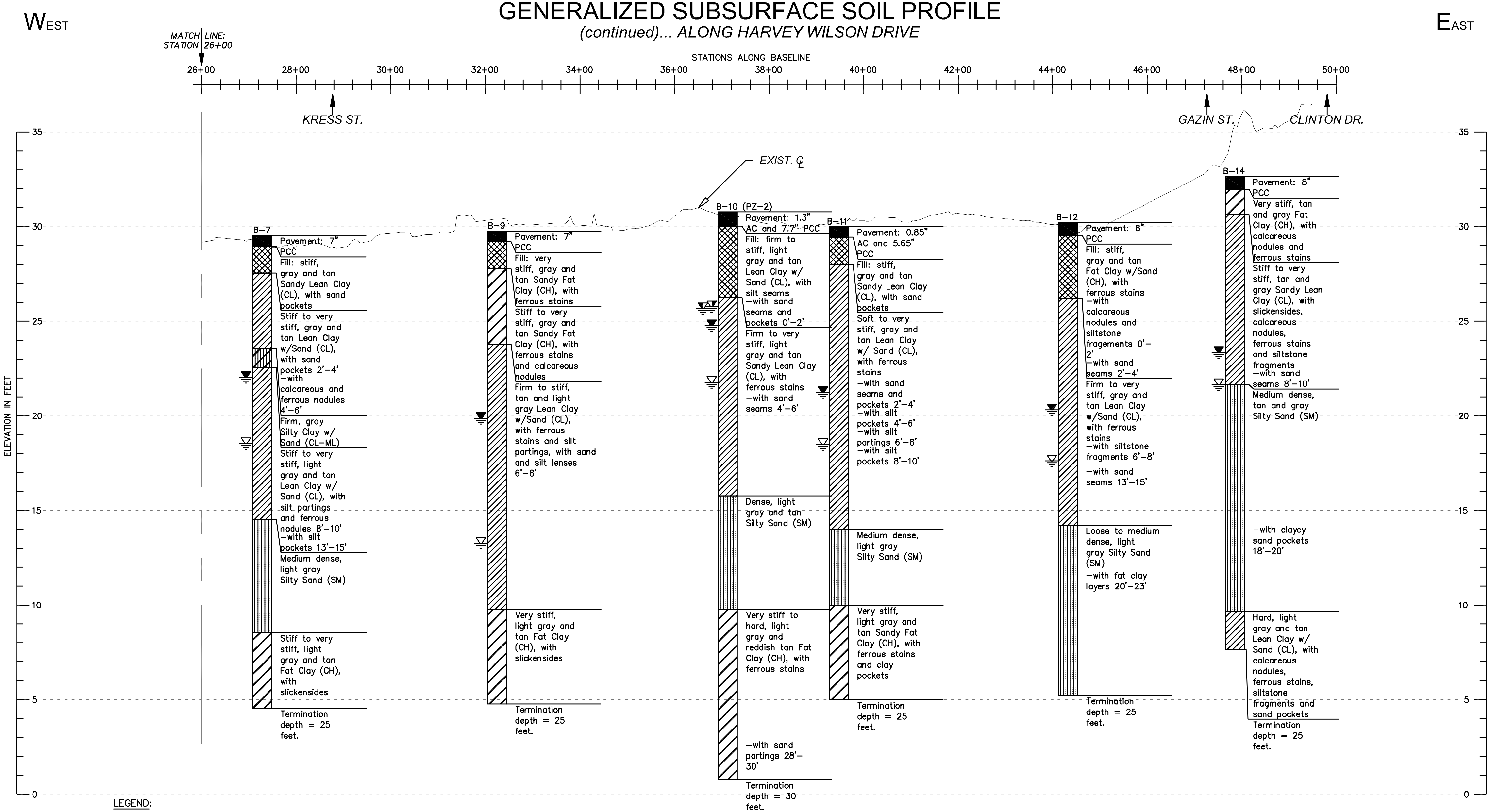
AVILES ENGINEERING CORPORATION

GENERALIZED SOIL PROFILE
HARVEY WILSON DRIVE RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.: G169-09	DATE: 10-20-09	SOURCE DRAWING PROVIDED BY: AVILES ENGINEERING CORP.
VERTICAL SCALE: 1" = 5' HORIZONTAL SCALE: 1" = 200'	DRAFTED BY: BpJ	PLATE NO.: PLATE B-3

GENERALIZED SUBSURFACE SOIL PROFILE

(continued)... ALONG HARVEY WILSON DRIVE



LEGEND:

Pavement	Silty low plasticity clay
Fill	Silty sand
High plasticity clay	Clayey sand
Low plasticity clay	

- Depth of groundwater (or cave–in) encountered during drilling
- Depth of groundwater after approx. 15 minutes
- Depth of groundwater after approx. 24 hours
- Depth of groundwater after approx. 30 days

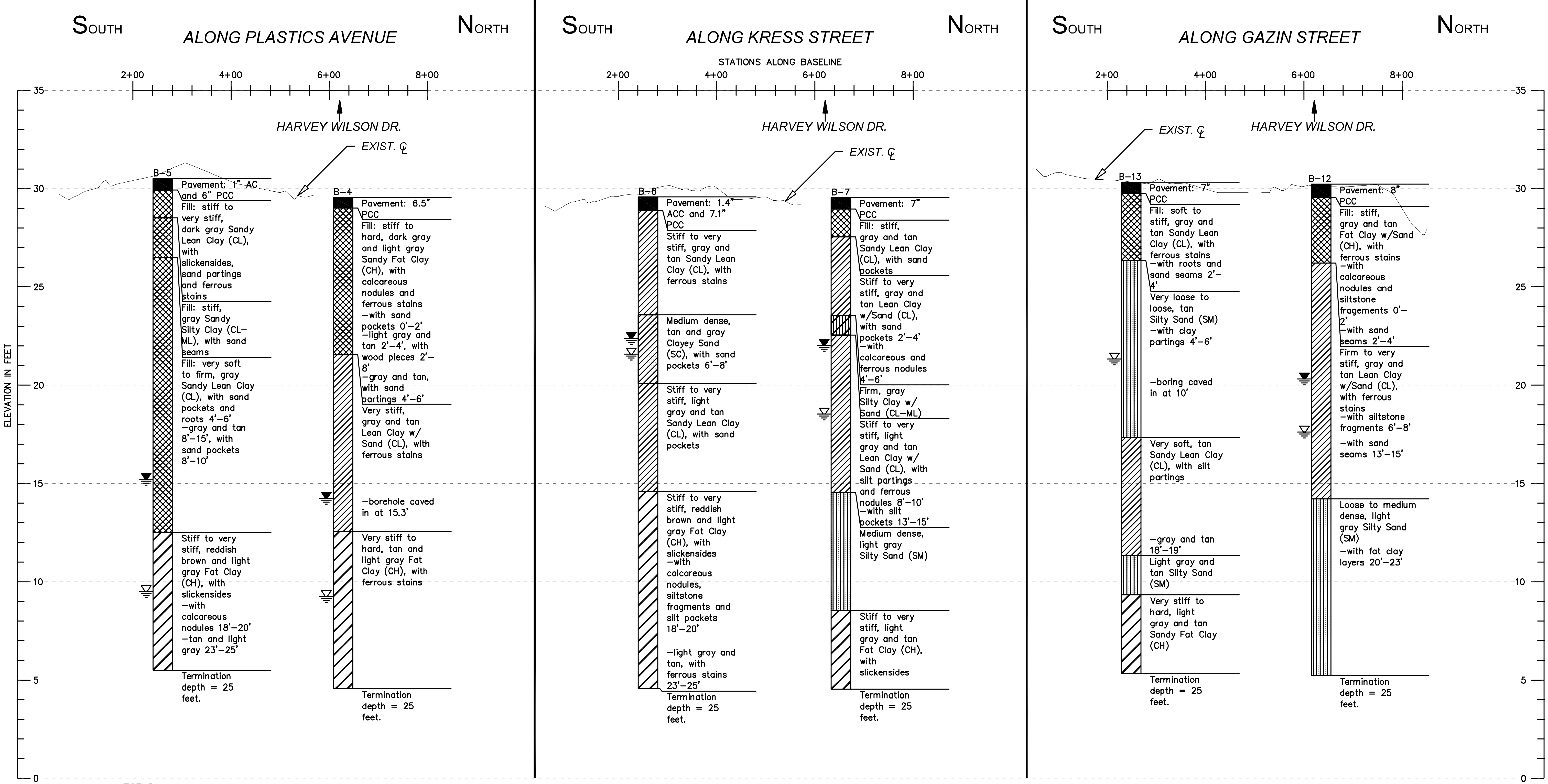
NOTE:
SOIL STRATIGRAPHY AND SECONDARY SOIL STRUCTURE (SUCH AS SEAMS, LAYERS, OR POCKETS OF SANDS, SILTS, SLICKENSIDES, AND FISSURES) THAT ARE DIFFERENT FROM WHAT WERE IDENTIFIED IN THE ACTUAL BORINGS MAY EXIST AWAY FROM THESE BORINGS.

AVILES ENGINEERING CORPORATION

GENERALIZED SOIL PROFILE
HARVEY WILSON DRIVE RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.: G169-09	DATE: 10-20-09	SOURCE DRAWING PROVIDED BY: AVILES ENGINEERING CORP.
VERTICAL SCALE: 1" = 5' / HORIZONTAL SCALE: 1" = 200'	DRAFTED BY: BpJ	PLATE NO.: PLATE B-4

GENERALIZED SUBSURFACE SOIL PROFILES



LEGEND:

Pavement	Silty low plasticity clay	Depth of groundwater (or cave-in) encountered during drilling
Fill	Silty sand	Depth of groundwater after approx. 15 minutes
High plasticity clay	Clayey sand	Depth of groundwater after approx. 24 hours
Low plasticity clay		Depth of groundwater after approx. 30 days

NOTE:

SOIL STRATIGRAPHY AND SECONDARY SOIL STRUCTURE (SUCH AS SEAMS, LAYERS, OR POCKETS OF SANDS, SILTS, SLICKENSIDES, AND FISSURES) THAT ARE DIFFERENT FROM WHAT WERE IDENTIFIED IN THE ACTUAL BORINGS MAY EXIST AWAY FROM THESE BORINGS.

AVILES ENGINEERING CORPORATION

GENERALIZED SOIL PROFILES
HARVEY WILSON DRIVE RECONSTRUCTION
WBS No. N-000733-0002-3
HOUSTON, TEXAS

AEC PROJECT NO.: G169-09	DATE: 10-20-09	SOURCE DRAWING PROVIDED BY: AVILES ENGINEERING CORP.
VERTICAL SCALE: 1" = 5' HORIZONTAL SCALE: 1" = 200'	DRAFTED BY: BpJ	PLATE NO.: PLATE B-5

AVILES
ENGINEERING CORP.

APPENDIX C

Plate C-1	Recommended Typical Geotechnical Soil Parameters
Plate C-2	Recommended OSHA Soil Types
Plate C-3	Load Coefficients for Pipe Loading
Plate C-4	Highway and Railroad Loads on Pipes Under Various Soil Cover

RECOMMENDED TYPICAL GEOTECHNICAL SOIL PARAMETERS
AEC G169-09

Soil Type		γ (pcf)	γ' (pcf)	E'_N (psi)	Short-Term					Long-Term				
					C (psf)	Φ (deg)	K_a	K_0	K_p	C (psf)	Φ (deg)	K_a	K_0	K_p
Fill	Fill: lime-stabilized clays	115	55	2000	3000	0	1.00	1.00	1.00	50	30	0.333	0.50	3.00
	Fill: firm CH/CL	115	55	200	500	0	1.00	1.00	1.00	40	16	0.57	0.72	1.76
	Fill: stiff CH/CL	120	60	400	1000	0	1.00	1.00	1.00	80	16	0.57	0.72	1.76
	Fill: very stiff CH/CL	125	65	900	1500	0	1.00	1.00	1.00	100	16	0.57	0.72	1.76
	Fill: ML/SM/SC/SP	115	55	300	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
Natural Soil	Soft CH/CL	115	55	100	250	0	1.00	1.00	1.00	0	12	0.66	0.79	1.53
	Firm CH/CL	118	58	300	500	0	1.00	1.00	1.00	50	14	0.61	0.76	1.64
	Stiff CH/CL	122	62	600	1000	0	1.00	1.00	1.00	150	16	0.57	0.72	1.76
	Very stiff CH/CL	125	65	1000	2000	0	1.00	1.00	1.00	200	18	0.53	0.69	1.89
	Hard CH/CL	125	65	2000	4000	0	1.00	1.00	1.00	300	20	0.49	0.66	2.04
	Loose ML/SM/SC/SP ($2 \le N_{SPT} \ge 7$)	115	55	300	0	26	0.39	0.56	2.56	0	26	0.39	0.56	2.56
	Loose to Med. dense ML/SM/SC ($8 \le N_{SPT} \ge 15$)	120	60	600	0	28	0.36	0.53	2.77	0	28	0.36	0.53	2.77
	Med. dense ML/SM/SC ($16 \le N_{SPT} \ge 30$)	120	60	1000	0	32	0.31	0.47	3.25	0	32	0.31	0.47	3.25
	Dense ML/SM/SC ($31 \le N_{SPT}$)	120	60	2000	0	34	0.28	0.44	3.53	0	34	0.28	0.44	3.53

Notes:
(1) CH = fat clay; CL = lean clay; ML = silt; SM = silty sand; SC = clayey sand; SP = poorly-graded sand.
(2) γ = unit weight for soil above water level; γ' = buoyant unit weight for soil below water level; γ_w = 62.4 pcf shall be used to compute hydrostatic pressure below water table.
(3) E'_N = modulus of soil reaction (stiffness) for initial flexible conduit deflection; C = cohesion; Φ = angle of internal friction.
(4) K_a = coefficient of active earth pressure; K_0 = coefficient of at-rest earth pressure; K_p = coefficient of passive earth pressure; level backfill.

AVILES ENGINEERING CORP.

OSHA SOIL CLASSIFICATION FOR EXCAVATION, TRENCHING AND SHORING

AEC Project No. G169-09

BORING NO.	DEPTH, FEET				
	0 - 5	5 - 10	10 - 15	15 - 20	Below 20
B-1	C	C	C	C	C
B-2	B	B	C	C	C
B-3	B	B	C	C	C
B-4	C	B	C	C	C
B-5	C	C	C	C	C
B-6	B	C	C	C	C
B-7	B	C	C	C	C
B-8	B	C	C	C	C
B-9	B	C	C	C	C
B-10	C	C	C	C	C
B-11	C	C	C	C	C
B-12	B	B	C	C	C
B-13	C	C	C	C	C
B-14	B	B	C	C	C

<u>OSHA Soil Types Definitions:</u>

A Clays with q_u=1.5 tsf or greater. No fissures or secondary structures, not subject to vibration, not been or will not be disturbed, not submerged or not have seeping water.

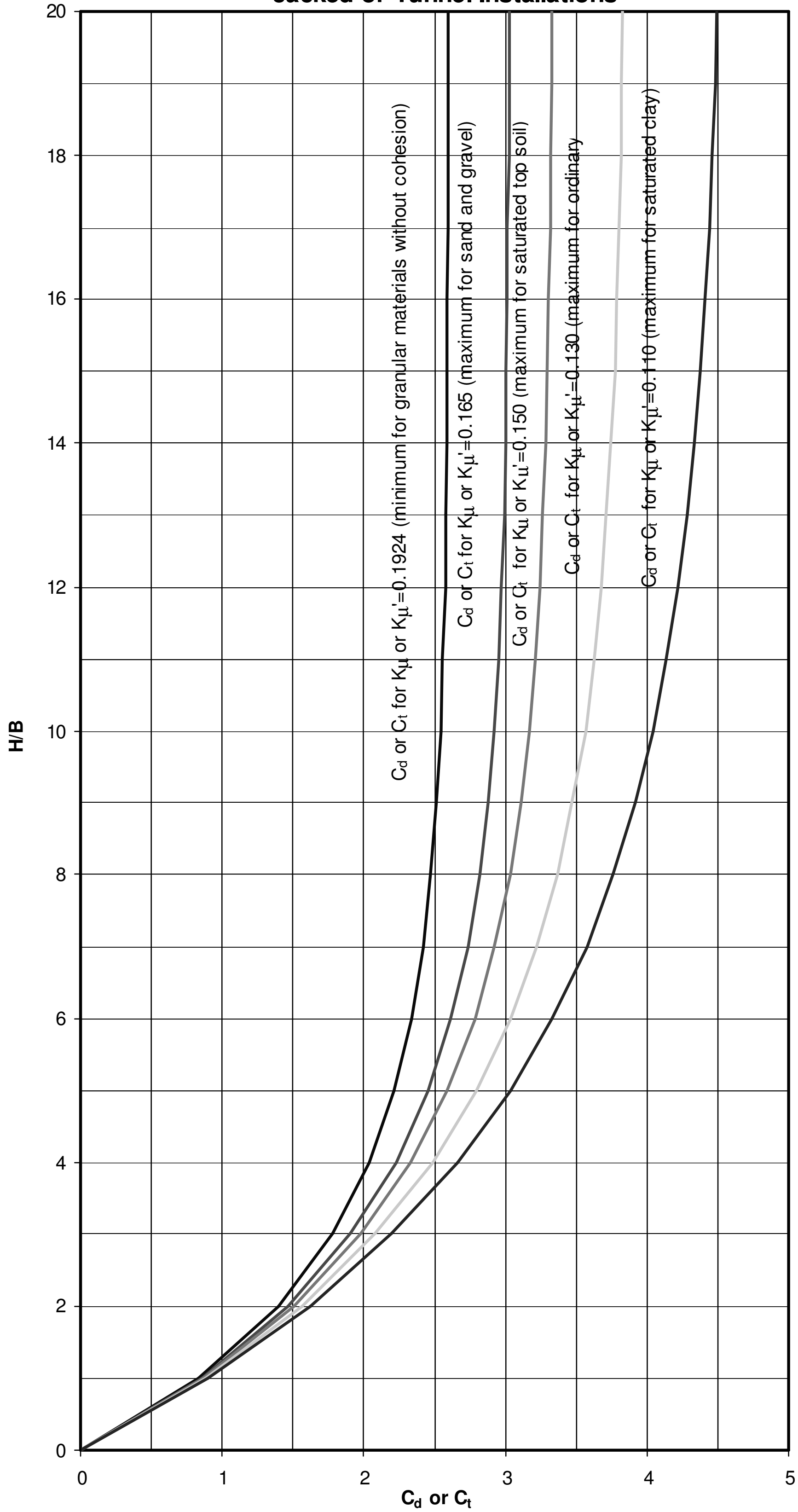
B Clays with q_u= between 0.5 and 1.5 tsf or clays with minor degree of slickensides or inclusion of granular material, angular gravel, silt, silt loam, dry and unstable rock, Type A soils with the above-mentioned exceptions. Shall not be submerged.

C Clays with q_u= less than 0.5 tsf or clays with significant amount of slickensides or with significant granular secondary granular material; or granular soils; soils which are submerged, saturated and has seepage; soils that will be subjected to disturbance.

<u>Notes</u>:

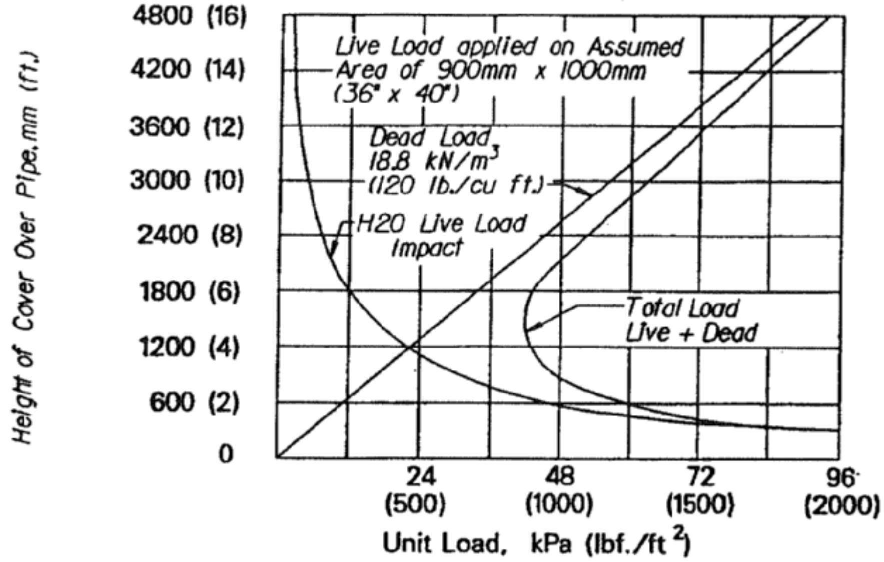
1. q_u is the unconfined compressive strength (or two times the cohesion) of a clay soil.
2. The trenching requirements shall meet the latest OSHA specified standards.
3. Use the weakest soil classification if a weaker soil layer underlies a stronger one.
4. Braced excavations deeper than 20 feet should be designed by a Professional Engineer.

Load Coefficient (C_d or C_t) for Conduits in Trench or Jacked or Tunnel Installations

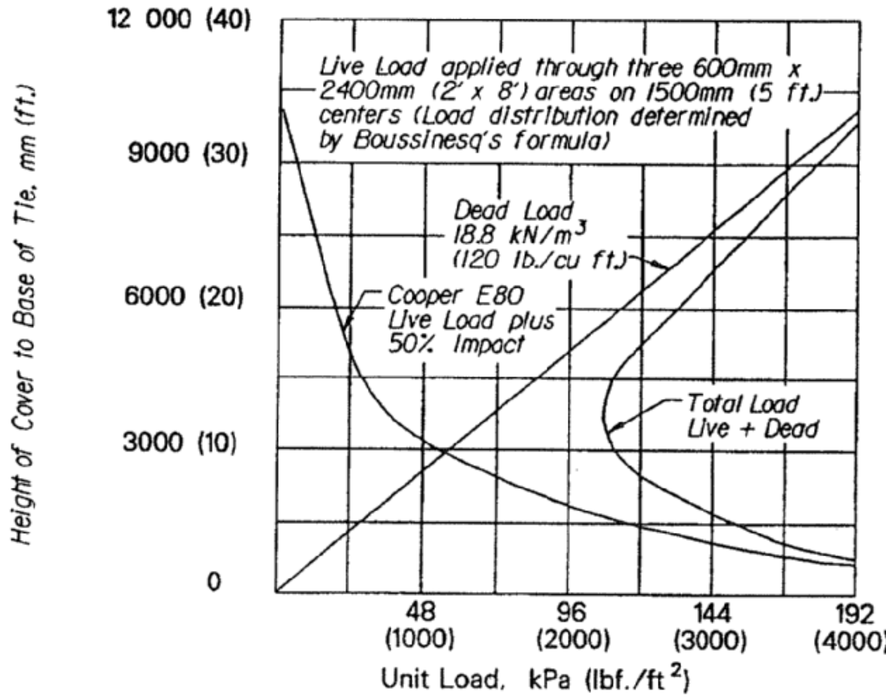


Reference: American Water Works Association, "Concrete Pressure Pipes", AWWA Manual M9.

HIGHWAY AND RAILROAD LOADS ON PIPES UNDER VARIOUS SOIL COVER



Combined H20 highway live load and dead load is a minimum at about 1500mm (5 ft.) of cover, applied through a pavement 300mm (1 ft.) thick.



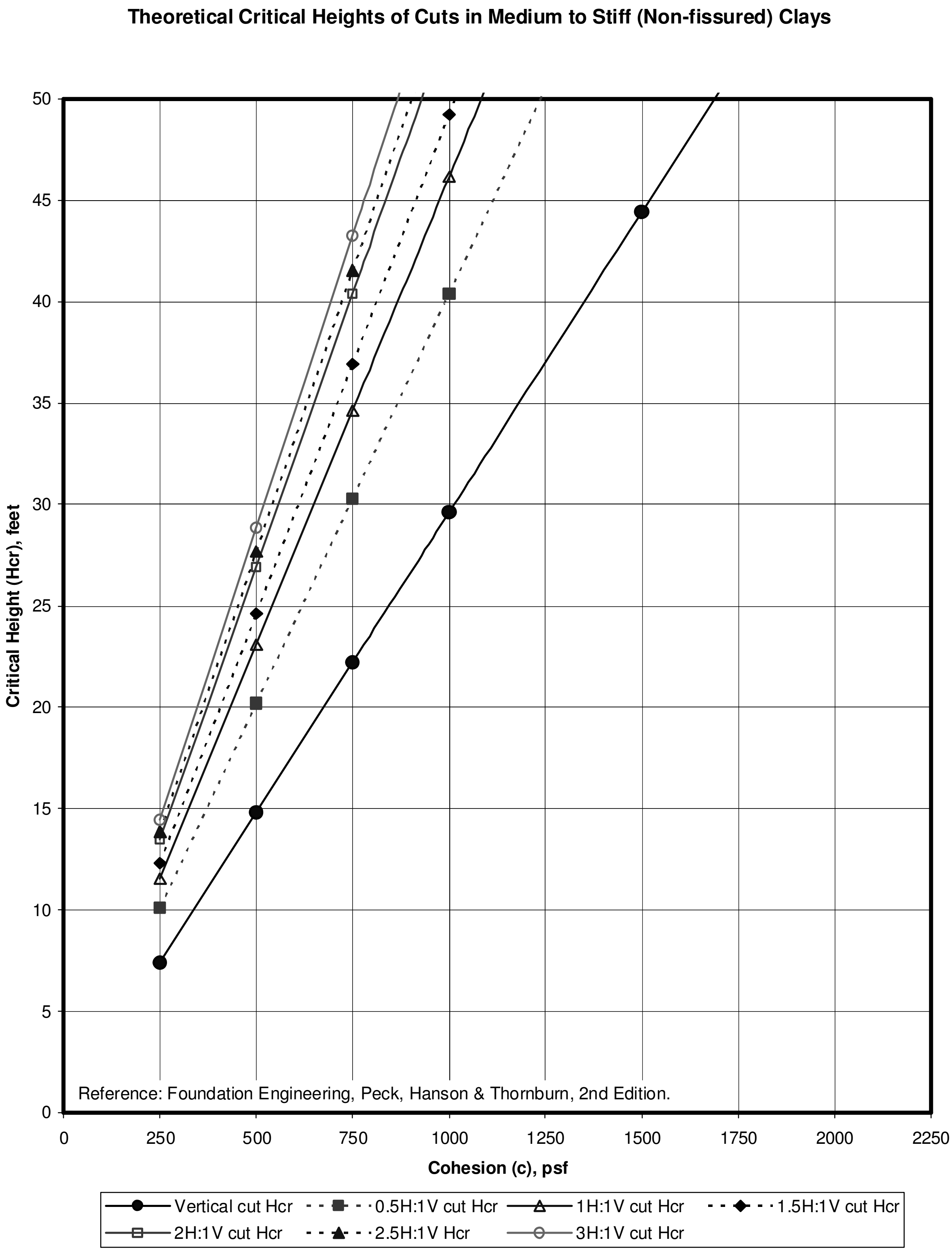
Railroad live load, Cooper E80, combined with dead load is a minimum at about 3600mm (12 ft.) Load is applied through three 600mm x 2400mm (2x8 ft.) areas on 1500mm (5 ft.) centers.

Reference: U.S. Army Corps of Engineers, (1997), "Conduits, Culverts, and Pipes", Engineer Manual 1110-2-2902, Page 5-3

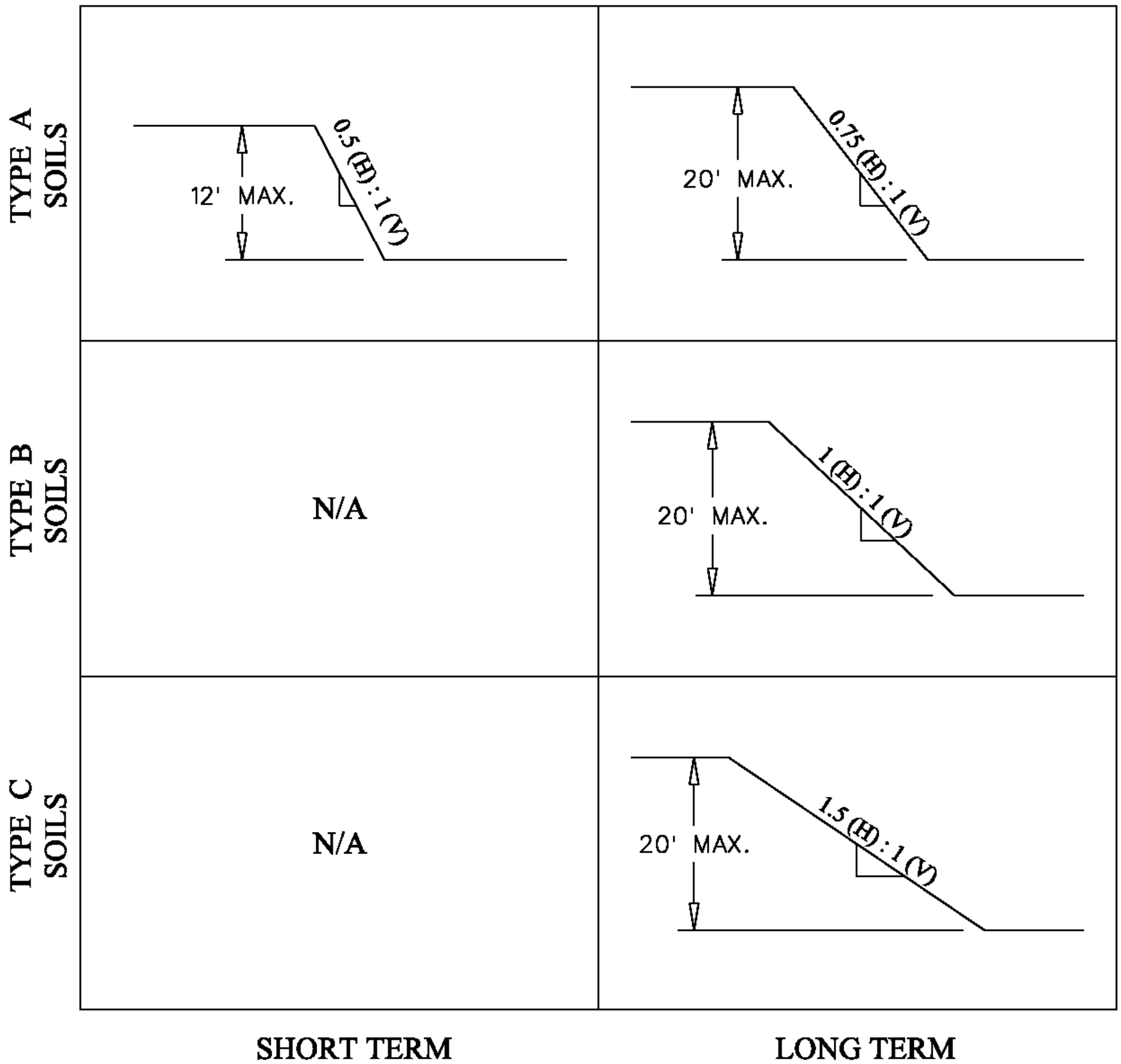
AVILES ENGINEERING CORP.

APPENDIX D

Plate D-1	Critical Heights of Cuts in Nonfissured Clays
Plate D-2	Maximum Allowable Slopes
Plate D-3	A Combination of Bracing and Open Cuts
Plate D-4	Pressure Envelope for Braced Cuts in Soft to Medium Clays
Plate D-5	Pressure Envelope for Braced Cuts in Stiff to Hard Clays
Plate D-6	Pressure Envelope for Braced Cuts in Sands
Plate D-7	Bottom Stability for Braced Excavation in Clay
Plate D-8	Buoyant Uplift Resistance for Buried Structures
Plate D-9	Influence Zone of Tunneling or Boring
Plate D-10	Thrust Force Calculation
Plate D-11	Thrust Force Example Calculation


Theoretical Critical Heights of Cuts in Medium to Stiff (Non-fissured) Clays
Critical Height (Hcr), feet
Cohesion (c), psf
0
5
10
15
20
25
30
35
40
45
50
0
250
500
750
1000
1250
1500
1750
2000
2250
Reference: Foundation Engineering, Peck, Hanson & Thornburn, 2nd Edition.
Vertical cut Hcr
0.5H:1V cut Hcr
1H:1V cut Hcr
1.5H:1V cut Hcr
2H:1V cut Hcr
2.5H:1V Hcr
3H:1V cut Hcr

MAXIMUM ALLOWABLE SLOPES



NOTES:

(1) For Type A soils, a short term maximum allowable slope of 0.5 (H) : 1 (V) is allowed in excavations that are 12 feet or less in depth; short term (24 hours or less) maximum allowable slopes for excavations greater than 12 feet in depth shall be 0.75 (H) : 1 (V).

(2) Maximum depth for above slopes is 20 feet. For slopes deeper than 20 feet, trench protection should be designed by the Contractor's professional engineer.

Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

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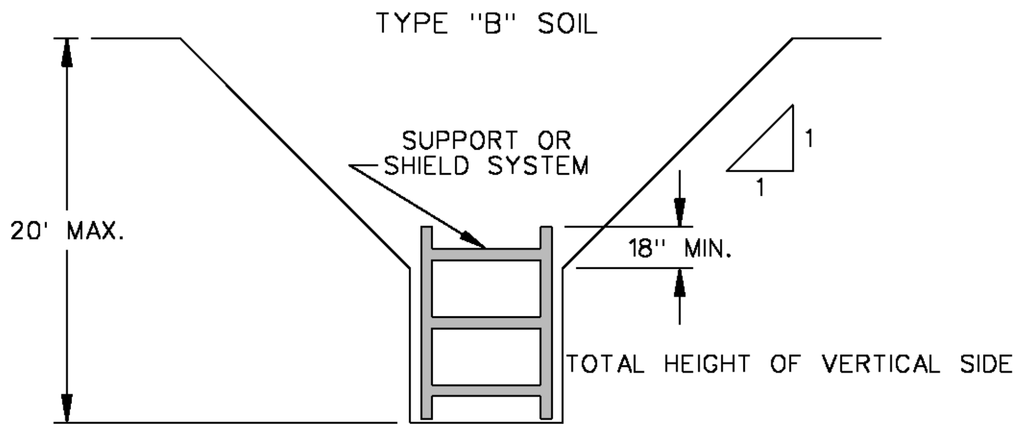
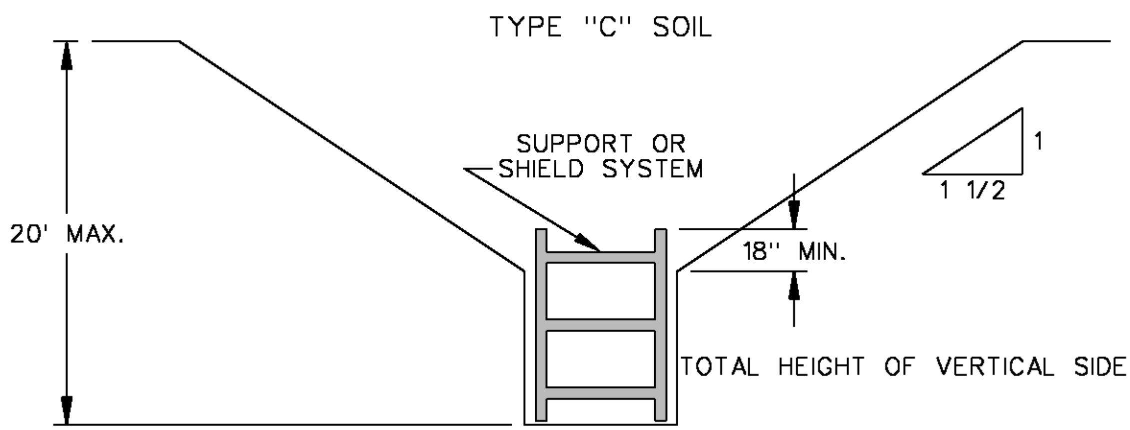
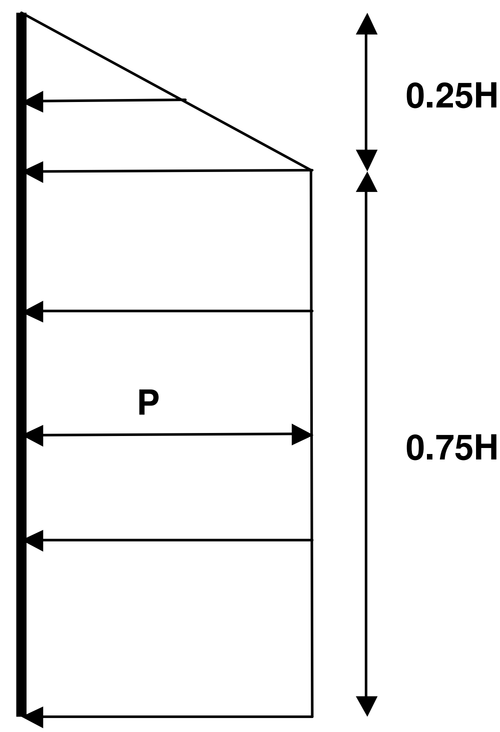
A COMBINATION OF BRACING AND OPEN CUTS





Reference: OSHA, Safety and Health Regulations for Construction, 1926 Subpart P.

PRESSURE ENVELOPE FOR BRACED CUTS IN SOFT TO MEDIUM CLAYS



$P = \gamma H - 4c$

or

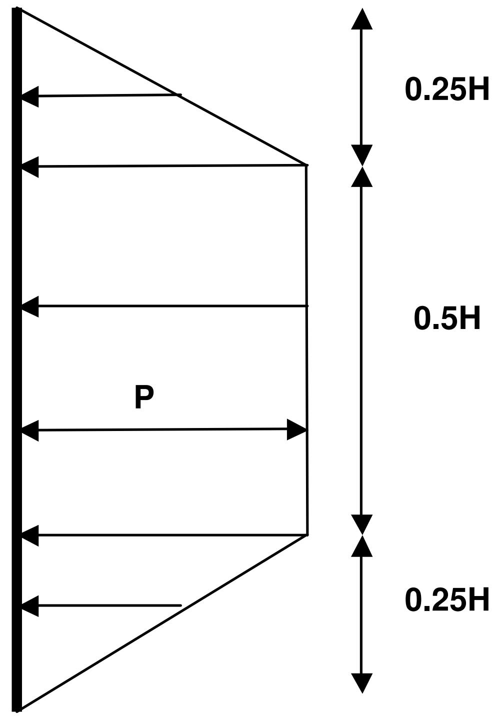
$= \gamma H - 1.6c$ (if soils at or below bottom of excavation have $c \leq 500$ psf)

Where,
- P = Lateral earth pressure, psf.
- γ = Unit weight of clays, use 130 pcf.
- H = Total height of the cut, feet.
- c = Undrained shear strength of clay, psf.

Notes:
1. This pressure diagram is applicable for the condition : $\gamma H/c > 4$
2. Surcharge and water pressure are not included.

Sources:
1) Naval Facilities Engineering Command (1986), "Foundations & Earth Structures", Pg 7.2-100.
2) R.B. Peck (1969), "Deep Excavations and Tunneling in Soft Ground," Proc. 7th Int. Conf. Soil Mech., Mexico, Pg 225-290.

PRESSURE ENVELOPE FOR BRACED CUTS IN STIFF TO HARD CLAYS



P = $0.3\gamma H$ (short term condition)
= $0.4\gamma H$ (long term condition)

Where, P = Lateral earth pressure, psf.
γ = Unit weight of clays, use 130 pcf.
H = Total height of the cut, feet.
c = Undrained shear strength of clay, psf.

Notes: 1. This pressure diagram is applicable for the condition :
$\gamma H/c$ <= 4
2. Surcharge and water pressure are not included.

Sources:
1) Naval Facilities Engineering Command (1986), "Foundations & Earth Structures", Pg 7.2-100.
2) R.B. Peck (1969), "Deep Excavations and Tunneling in Soft Ground," Proc. 7th Int. Conf. Soil Mech., Mexico, Pg 225-290.

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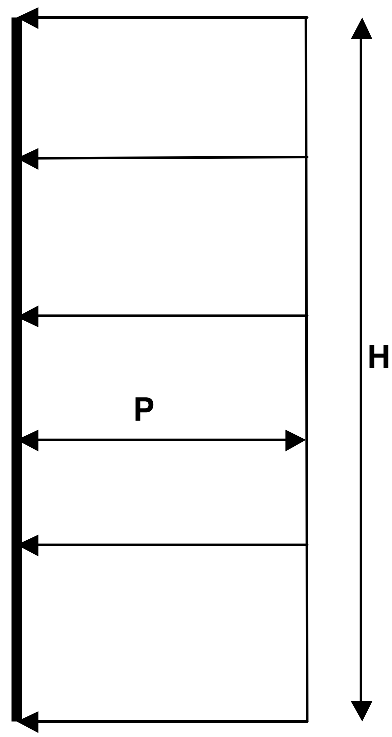
PRESSURE ENVELOPE FOR BRACED CUTS IN SANDS



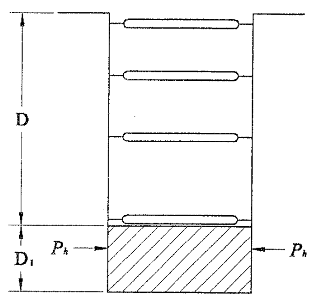
$P = 0.65\ \gamma H K_a$

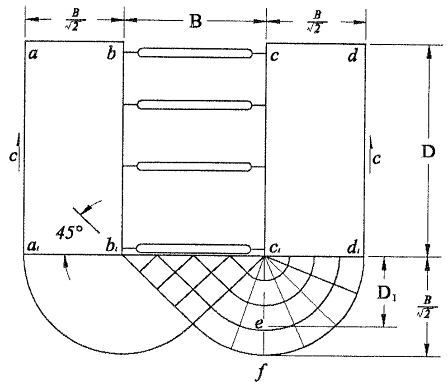
Where, P = Lateral earth pressure, psf.
γ = Unit weight of sands, use 120 pcf.
H = Total height of the cut, feet.
K_a = Active earth pressure coefficient = $\tan^2(45-\Phi/2) = 0.361$ for $\Phi=28^o$

Note: Surcharge and water pressure are not included.

Sources:
1) Naval Facilities Engineering Command (1986), "Foundations & Earth Structures", Pg 7.2-100.
2) R.B. Peck (1969), "Deep Excavations and Tunneling in Soft Ground," Proc. 7th Int. Conf. Soil Mech., Mexico, Pg 225-290.

BOTTOM STABILITY FOR BRACED EXCAVATION IN CLAY





Factor of safety against bottom heave in braced excavation in clay:

$$Fs = \frac{N_c C}{\gamma_1' D + q}$$

If Fs is <1.5, sheeting should be extended at least D_1 below the excavation bottom to achieve stability:

$$\text{Required } D_1 = \frac{1.5(\gamma_1' D + q) - N_c C}{\gamma_2'} \quad ; \quad D_1 \geq 5 \text{ ft.}$$

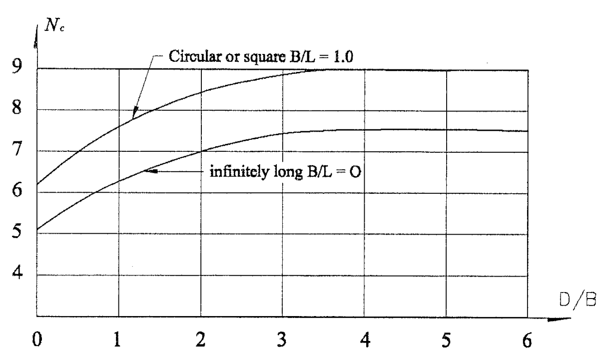
where,

where,	Nc	=	Coefficient depending on the dimension of excavation (see chart at the bottom)
	C	=	Undrained shear strength of the foundation clay soil at excavation bottom, psf
	γ_1'	=	Average effective unit weight of soils above excavation bottom (adjusted for groundwater as necessary), pcf
	γ_2'	=	Average effective unit weight of soils below excavation bottom and within sheeting embedment zone (assume groundwater level at bottom of excavation to be conservative), pcf
	D	=	Depth of excavation, ft.
	q	=	Surface surcharge load, psf
	B	=	Width of excavation, ft.

Pressure on buried length, P_h:

For $D_1 < 0.47B$: $P_h = 1.5D_1 (\gamma_1' D - 1.4 C D / B - 3.14 C)$

For $D_1 > 0.47B$: $P_h = 0.7 (\gamma_1' D B - 1.4 C D - 3.14 C B)$



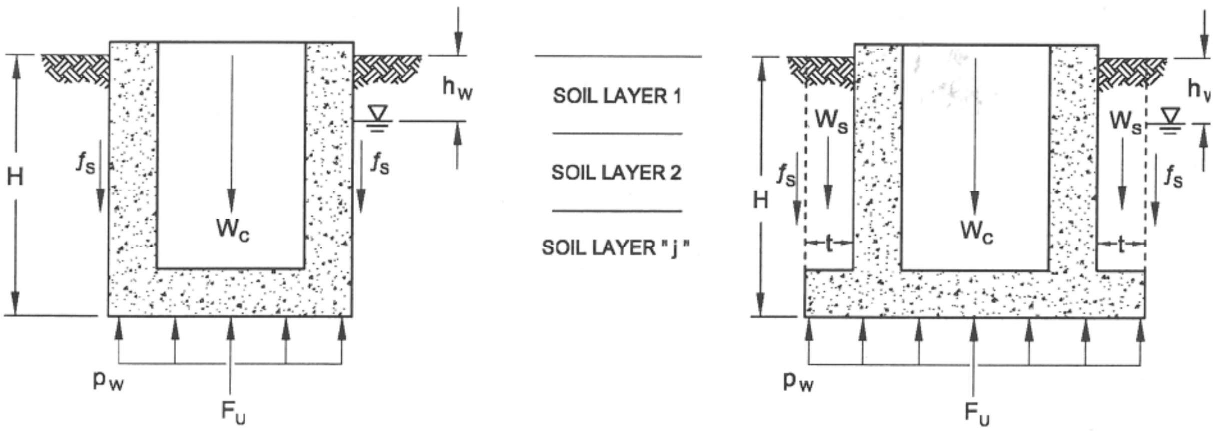
N_c rectangular = $(0.84 + 0.16B/L)N_c$ square

Reference: NAVFAC DM 7.2, "Foundation and Earth Structures Design Manual", Alexandria, Virginia

BUOYANT UPLIFT RESISTANCE FOR BURIED STRUCTURES

(a) WALL / SOIL FRICTION PLUS STRUCTURAL WEIGHT

(b) SOIL WEIGHT ABOVE BASE EXTENSIO



(a)

cohesive soils: $f_{s_j} = \alpha\ c_j \leqq 3{,}000$ psf

cohesionless soils: $f_{s_j} = 0.75\ K_s\ \sigma_{v_j}\ \tan\delta_j$

$$Q_s = P_s \sum f_{s_j}\ h_j$$

$$\frac{W_c}{S_{f_a}} + \frac{Q_s}{S_{f_b}} \geqq F_U$$

(b)

cohesive soils: $f_{s_j} = c_j \leqq 3{,}000$ psf

cohesionless soils: $f_{s_j} = 0.75\ K_s\ \sigma_{v_j}\ \tan\Phi_j$

$$Q_s = P_s \sum f_{s_j}\ h_j$$

$$\frac{W_c}{S_{f_a}} + \frac{Q_s}{S_{f_b}} + \frac{W_s}{S_{f_c}} \geqq F_U$$

Where:

- A_B = area of base, sq. ft.
- H = buried height of structure, ft.
- h_w = depth to water table, ft.
- p_w = $\gamma_w\,(H - h_w)$, unit hydrostatic uplift, psf.
- γ_w = 62.4 pcf, unit weight of water
- F_U = $p_w A_B$, hydrostatic uplift force, lbs.
- f_{s_j} = unit frictional resistance of soil layer "j", psf.
- c_j = undrained cohesion of soil layer "j", psf.
- α = 0.55, cohesion factor between soil and structure wall
- σ_{v_j} = effective overburden pressure at midpoint of soil layer "j", psf.
- δ_j = 0.75 Φ_j, friction angle between soil layer "j" and concrete wall, degrees
- Φ_j = internal angle of friction of soil layer "j", degrees
- K_s = 0.4, coefficient of lateral pressure
- h_j = thickness of soil layer "j", ft.
- j = 1, 2, ...
- P_s = perimeter of structure base, ft.
- Q_s = ultimate skin friction, lbs.
- W_c = weight of structure, lbs.
- W_s = weight of backfill above base extension, lbs.
- S_{f_a} = 1.1, factor of safety for dead weight of structure
- S_{f_b} = 3.0, factor of safety for soil / structure friction
- S_{f_c} = 1.5, factor of safety for soil weight above base extension
- t = width of base extension, ft.

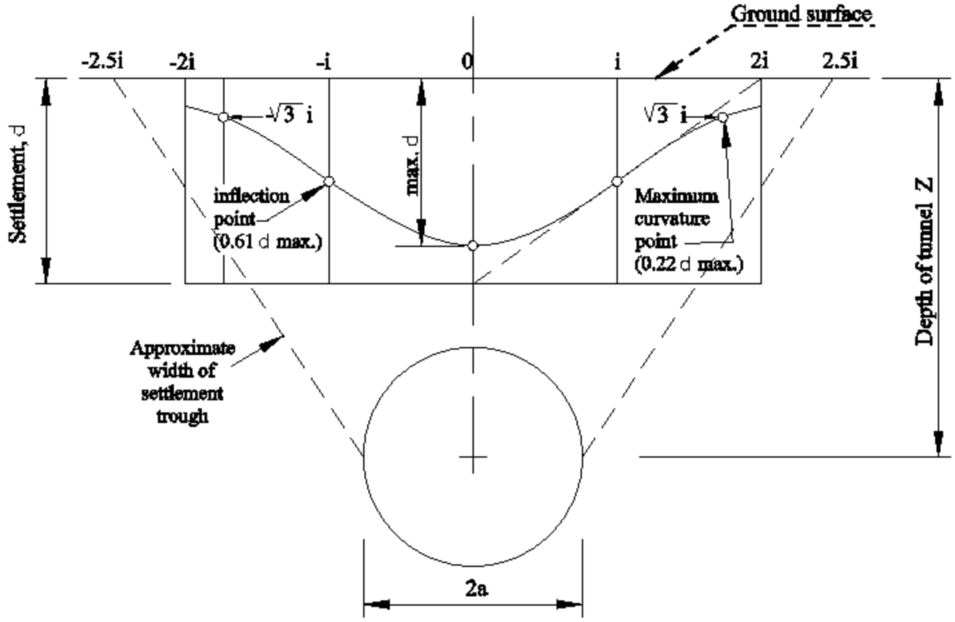
NOTE: neglect f_s in upper 5 feet for expansive clay with a plasticity index > 20.

Reference:

1) American Concrete Pipe Association, (1996), Manhole Floatation

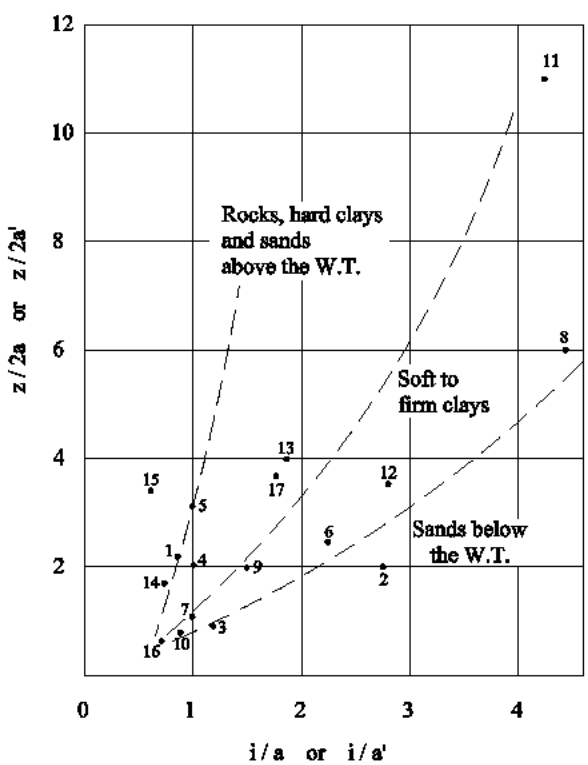
2) O'Neill, M.W., and Reese, L.C., (1999), "Drilled Shafts: Construction Procedures and Design Methods", FHWA–IF–99–025

Relation between the Width of the Surface Depression (i/a) and the Depth of the Cavity (z/a) for Tunnels



Volume of depression = 2.5id max.

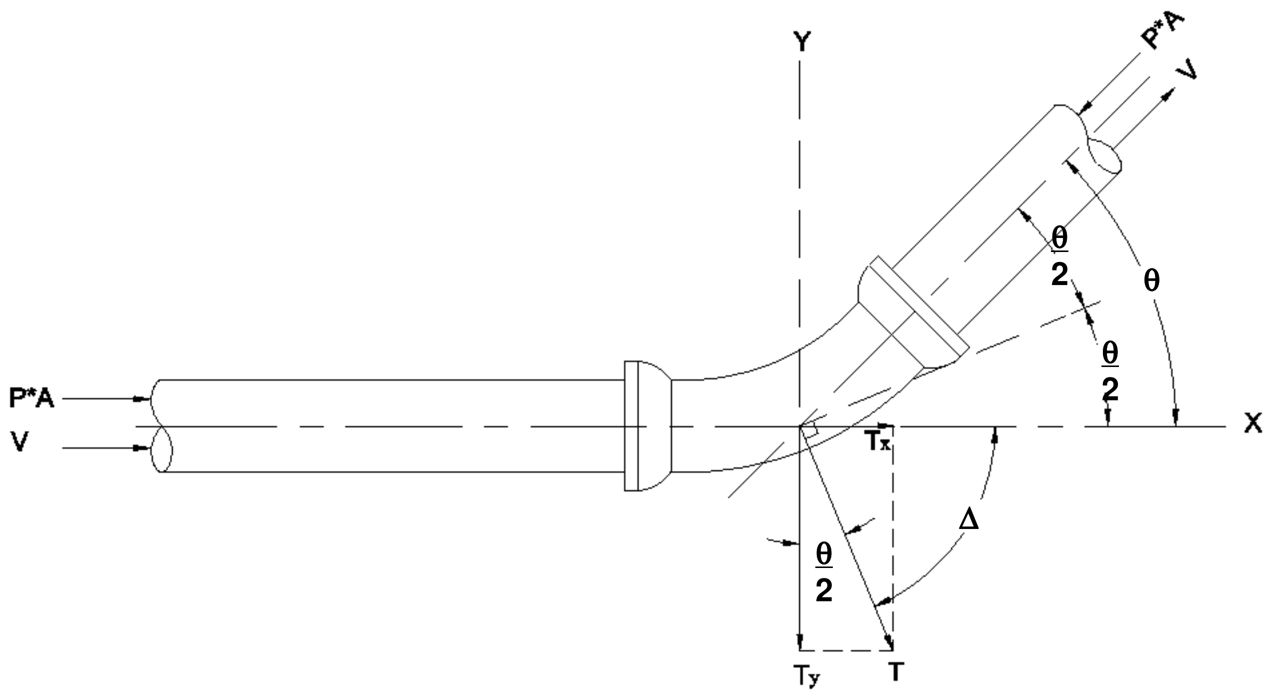
(a)



(b)

Reference: Peck, R. B. (1969) "Deep Excavations and Tunneling in Soft Ground," Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, State of the Art Volume, pp. 225-290.

THRUST FORCE AT A PIPE BEND



$$T = 2\,P\,A\sin\frac{\theta}{2}$$

$$T_x = P\,A\,(1 - \cos\theta)$$

$$T_y = P\,A\sin\theta$$

Where:

T	=	Resultant thrust force, lbs
T_x	=	Resultant thrust force component along x-axis, lbs
T_y	=	Resultant thrust force component along y-axis, lbs
P	=	Maximum sustain pressure of fluid in pipe, psi
A	=	Cross-section area of pipe, square inches
D	=	Inside diameter of pipe, inches
θ	=	Angle of the pipe bend, degrees
Δ	=	Angle between x-axis and resultant force
	=	$\tan^{-1}(T_y/T_x)$, degrees
V	=	Fluid velocity

Source: American Water Works Association, "Concrete Pressure Pipes", AWWA Manual M9.

Thrust Force At A Pipe Bend Example Calculation

Given:

D	=	24 inches
P	=	200 psi
θ	=	60 degrees

Find: T, T_x and T_y

A	=	$\pi D^2/4$	=	452.39 in^2
T	=	2 * 200 * 452.39 * sin(60 º /2)	=	90,478 lbs
T_x	=	200 * 452.39 * (1 – cos 60 º)	=	45,239 lbs
T_y	=	200 * 452.39 * sin 60º	=	78,356 lbs

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APPENDIX E

Well and Plugging Reports for Two Piezometers Installed In This Project
As Reported to Texas Department of Licensing and Regulation (TDLR)

Send original copy by certified return receipt requested mail to:

TDLR
P.O. Box 12157
Austin, TX 78711
(512) 463-7880

State of Texas
PLUGGING REPORT

(This form must be completed and filed with the TDLR within 30 days following the date the well is plugged as required by current statutory law.)

A. WELL IDENTIFICATION AND LOCATION DATA

1) OWNER City of Houston (Name) ADDRESS 611 Walker Floor 14 (Street or RFD) Houston (City) Tx (State) 77002 (Zip)

2) ADDRESS OF WELL: County Harvey Wilson Rd (Street, RFD or other) Houston (City) Tx (State) 77020 (Zip) GRID # 65-14-8

3) OWNER'S WELL NO: B-2

4) WELL TYPE (Check): ☐ Water ☐ Monitor ☐ Injection ☒ Piezometer

5)

x

N

Driller, Pump Installer, or Landowner performing the plugging operations must locate and identify the location of the well within a specific grid on a full scale-gridded County map available from the Texas Water Development Board. The location of the well should be denoted within the grid by placing a corresponding dot in the grid to the right. The legal description section below is optional.

☐ LEGAL DESCRIPTION: NA

Section No. ______ Block No. ______ Township ______

Abstract No. ______ Survey Name ______

Distance and direction from two intersecting section lines or survey lines: ______

B. HISTORICAL DATA ON WELL TO BE PLUGGED (IF AVAILABLE)

6) Driller Eddie Van Antwerp License Number 2903M City Houston

7) Drilled 08/25 (Mo/Day) 2009 (Year); 8) Diameter of hole 4 inches; 9) Total depth of well 20 feet.

C. CURRENT PLUGGING DATA

10) Date well plugged 10/05, 2009 (Year).

11) Sketch of well: Using space at right, show method of plugging the well, including all casing and cemented intervals.

12) Name of Driller/Pump Installer actually performing the plugging operations

Eddie Van Antwerp

License Number 2903M

13) Casing and cementing data relative to the plugging operations:

DIAMETER (Inches)	CASING LEFT IN WELL FROM (feet)	CASING LEFT IN WELL TO (feet)
2	0	20

CEMENT / BENTONITE PLUG(S) PLACED IN WELL FROM (feet)	TO (feet)	SACK(S) OF CEMENT USED
0	20	2

1) **Tried to pull the well, top section broke off.**
2) **Grouted the entire length with cement/bentonite grout.**

D. VALIDATION OF INFORMATION INCLUDED IN FORM

I certify that I plugged this well (or the well was plugged under my supervision) and that each and all of the statements herein are true and correct. I understand that failure to complete items 1 thru 13 will result in the report(s) being returned for completion and resubmittal.

Company or Individual's Name *(type or print)* Van & Sons Drilling Service, Inc

Address: Street or RFD 319 John Alber City Houston State Tx Zip 77076

Signatures: [signature] (Licensed Driller/Pump Installer) 10/08/09 (Date) ______ (Trainee/Apprentice) ______ (Date)

TDLR FORM 004WWD (4/98)

Copies to TNRCC, Well Owner, and Licensed Well Driller/Pump Installer

Forms On-A-Disk, Inc. · Dallas, Texas (214) 340-9429

Send original copy by certified return receipt requested mail to:

TDLR
P.O. Box 12157
Austin, TX 78711
(512) 463-7880

State of Texas
PLUGGING REPORT

(This form must be completed and filed with the TDLR within 30 days following the date the well is plugged as required by current statutory law.)

A. WELL IDENTIFICATION AND LOCATION DATA

1) OWNER City of Houston (Name) ADDRESS 611 Walker Floor 14 (Street or RFD) Houston (City) Tx (State) 77002 (Zip)

2) ADDRESS OF WELL: County Harvey Wilson Rd (Street, RFD or other) Houston (City) Tx (State) 77020 (Zip) GRID # 65-14-8

3) OWNER'S WELL NO: B-10

4) WELL TYPE (Check): ☐ Water ☐ Monitor ☐ Injection ☒ Piezometer

5)

x

N

Driller, Pump Installer, or Landowner performing the plugging operations must locate and identify the location of the well within a specific grid on a full scale-gridded County map available from the Texas Water Development Board. The location of the well should be denoted within the grid by placing a corresponding dot in the grid to the right. The legal description section below is optional.

☐ LEGAL DESCRIPTION: NA

Section No. ______ Block No. ______ Township ______

Abstract No. ______ Survey Name ______

Distance and direction from two intersecting section lines or survey lines: ______

B. HISTORICAL DATA ON WELL TO BE PLUGGED (IF AVAILABLE)

6) Driller Eddie Van Antwerp License Number 2903M City Houston

7) Drilled 08/25 (Mo/Day) 2009 (Year); 8) Diameter of hole 4 inches; 9) Total depth of well 20 feet.

C. CURRENT PLUGGING DATA

10) Date well plugged 10/05, 2009 (Year).

11) Sketch of well: Using space at right, show method of plugging the well, including all casing and cemented intervals.

1) **Tried to pull the well, top section broke off.**
2) **Grouted the entire length with cement/bentonite grout.**

12) Name of Driller/Pump Installer actually performing the plugging operations Eddie Van Antwerp

License Number 2903M

13) Casing and cementing data relative to the plugging operations:

DIAMETER (Inches)	CASING LEFT IN WELL FROM (feet)	CASING LEFT IN WELL TO (feet)
2	0	25

CEMENT / BENTONITE PLUG(S) PLACED IN WELL FROM (feet)	CEMENT / BENTONITE PLUG(S) PLACED IN WELL TO (feet)	SACK(S) OF CEMENT USED
0	25	2

D. VALIDATION OF INFORMATION INCLUDED IN FORM

I certify that I plugged this well (or the well was plugged under my supervision) and that each and all of the statements herein are true and correct. I understand that failure to complete items 1 thru 13 will result in the report(s) being returned for completion and resubmittal.

Company or Individual's Name *(type or print)* Van & Sons Drilling Service, Inc

Address: Street or RFD 319 John Alber City Houston State Tx Zip 77076

Signatures: [signature] Licensed Driller/Pump Installer 10/08/09 Date ______ Trainee/Apprentice ______ Date

TDLR FORM 004WWD (4/98)

Copies to TNRCC, Well Owner, and Licensed Well Driller/Pump Installer

Forms On-A-Disk, Inc. · Dallas, Texas (214) 340-9429

Type or print on this form a list of wells for which you are submitting "Well Report" forms.

STATE OF TEXAS

STATE WELL REPORT SUBMISSION FORM

Send original copy, along with the original copies of your State Well Reports, by certified mail to:
TDLR
P.O. Box 12157
Austin, Texas 78711

DATE PLUGGED	WELL OWNER	COUNTY
B-2 & B-10 on 10/05/2009	City of Houston	Harris
	611 Walker	
	Floor 14	
	Houston, Tx 77002-4903	

Eddie Van Antwerp	2903M	10/08/09
LICENSED DRILLER/PUMP INSTALLER (Please type or print)	LICENSE NUMBER	DATE

Send original copy by certified return receipt requested mail to:

TDLR
P.O. Box 12157
Austin, TX 78711
(512) 463-7880

State of Texas

PLUGGING REPORT

(This form must be completed and filed with the TDLR within 30 days following the date the well is plugged as required by current statutory law.)

A. WELL IDENTIFICATION AND LOCATION DATA

1) OWNER **City of Houston** (Name) ADDRESS **611 Walker Floor 14** (Street or RFD) **Houston** (City) **Tx** (State) **77002** (Zip)

2) ADDRESS OF WELL: County **Harvey Wilson Rd** (Street, RFD or other) **Houston** (City) **Tx** (State) **77020** (Zip) GRID # **65-14-8**

3) OWNER'S WELL NO: B-2

4) WELL TYPE (Check): ☐ Water ☐ Monitor ☐ Injection ☒ Piezometer

5)

x

N

Driller, Pump Installer, or Landowner performing the plugging operations must locate and identify the location of the well within a specific grid on a full scale-gridded County map available from the Texas Water Development Board. The location of the well should be denoted within the grid by placing a corresponding dot in the grid to the right. The legal description section below is optional.

☐ LEGAL DESCRIPTION: NA

Section No. ______ Block No. ______ Township ______

Abstract No. ______ Survey Name ______

Distance and direction from two intersecting section lines or survey lines: ______

B. HISTORICAL DATA ON WELL TO BE PLUGGED (IF AVAILABLE)

6) Driller **Eddie Van Antwerp** License Number **2903M** City **Houston**

7) Drilled **08/25** (Mo/Day) **2009** (Year); 8) Diameter of hole **4** inches; 9) Total depth of well **20** feet.

C. CURRENT PLUGGING DATA

10) Date well plugged **10/05**, **2009** (Year).

11) Sketch of well: Using space at right, show method of plugging the well, including all casing and cemented intervals.

1) Tried to pull the well, top section broke off.
2) Grouted the entire length with cement/bentonite grout.

12) Name of Driller/Pump Installer actually performing the plugging operations

Eddie Van Antwerp

License Number **2903M**

13) Casing and cementing data relative to the plugging operations:

DIAMETER (Inches)	CASING LEFT IN WELL FROM (feet)	CASING LEFT IN WELL TO (feet)
2	0	20

CEMENT / BENTONITE PLUG(S) PLACED IN WELL FROM (feet)	CEMENT / BENTONITE PLUG(S) PLACED IN WELL TO (feet)	SACK(S) OF CEMENT USED
0	20	2

D. VALIDATION OF INFORMATION INCLUDED IN FORM

I certify that I plugged this well (or the well was plugged under my supervision) and that each and all of the statements herein are true and correct. I understand that failure to complete items 1 thru 13 will result in the report(s) being returned for completion and resubmittal.

Company or Individual's Name *(type or print)* **Van & Sons Drilling Service, Inc**

Address: Street or RFD **319 John Alber** City **Houston** State **Tx** Zip **77076**

Signatures: [signature] Licensed Driller/Pump Installer **10/08/09** Date ______ Trainee/Apprentice ______ Date

Copies to TNRCC, Well Owner, and Licensed Well Driller/Pump Installer

Send original copy by certified return receipt requested mail to:

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Austin, TX 78711
(512) 463-7880

State of Texas
PLUGGING REPORT

(This form must be completed and filed with the TDLR within 30 days following the date the well is plugged as required by current statutory law.)

A. WELL IDENTIFICATION AND LOCATION DATA

1) OWNER City of Houston (Name) ADDRESS 611 Walker Floor 14 (Street or RFD) Houston (City) Tx (State) 77002 (Zip)

2) ADDRESS OF WELL: County Harvey Wilson Rd (Street, RFD or other) Houston (City) Tx (State) 77020 (Zip) GRID # 65-14-8

3) OWNER'S WELL NO: B-10

4) WELL TYPE (Check): ☐ Water ☐ Monitor ☐ Injection ☒ Piezometer

5)

x

N

Driller, Pump Installer, or Landowner performing the plugging operations must locate and identify the location of the well within a specific grid on a full scale-gridded County map available from the Texas Water Development Board. The location of the well should be denoted within the grid by placing a corresponding dot in the grid to the right. The legal description section below is optional.

☐ LEGAL DESCRIPTION: NA

Section No. ______ Block No. ______ Township ______

Abstract No. ______ Survey Name ______

Distance and direction from two intersecting section lines or survey lines: ______

B. HISTORICAL DATA ON WELL TO BE PLUGGED (IF AVAILABLE)

6) Driller Eddie Van Antwerp License Number 2903M City Houston

7) Drilled 08/25 (Mo/Day) 2009 (Year); 8) Diameter of hole 4 inches; 9) Total depth of well 20 feet.

C. CURRENT PLUGGING DATA

10) Date well plugged 10/05, 2009 (Year).

11) Sketch of well: Using space at right, show method of plugging the well, including all casing and cemented intervals.

1) Tried to pull the well, top section broke off.
2) Grouted the entire length with cement/bentonite grout.

12) Name of Driller/Pump Installer actually performing the plugging operations

Eddie Van Antwerp

License Number 2903M

13) Casing and cementing data relative to the plugging operations:

DIAMETER (Inches)	CASING LEFT IN WELL FROM (feet)	TO (feet)
2	0	25

CEMENT / BENTONITE PLUG(S) PLACED IN WELL FROM (feet)	TO (feet)	SACK(S) OF CEMENT USED
0	25	2

D. VALIDATION OF INFORMATION INCLUDED IN FORM

I certify that I plugged this well (or the well was plugged under my supervision) and that each and all of the statements herein are true and correct. I understand that failure to complete items 1 thru 13 will result in the report(s) being returned for completion and resubmittal.

Company or Individual's Name *(type or print)* Van & Sons Drilling Service, Inc

Address: Street or RFD 319 John Alber City Houston State Tx Zip 77076

Signatures: E. J. Van Antwerp (Licensed Driller/Pump Installer) 10/08/09 (Date) ______ (Trainee/Apprentice) ______ (Date)

TDLR FORM 004WWD (4/98)

Copies to TNRCC, Well Owner, and Licensed Well Driller/Pump Installer

Forms On-A-Disk, Inc. · Dallas, Texas (214) 340-9429

Type or print on this form a list of wells for which you are submitting "Well Report" forms.

STATE OF TEXAS

STATE WELL REPORT SUBMISSION FORM

Send original copy, along with the original copies of your State Well Reports, by certified mail to:
TDLR
P.O. Box 12157
Austin, Texas 78711

DATE PLUGGED	WELL OWNER	COUNTY
B-2 & B-10 on 10/05/2009	City of Houston	Harris
	611 Walker	
	Floor 14	
	Houston, Tx 77002-4903	

Eddie Van Antwerp
LICENSED DRILLER/PUMP INSTALLER
(Please type or print)

2903M
LICENSE NUMBER

10/08/09
DATE